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Tests of Concrete Columns Reinforced with Structural Shapes

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TESTS OF CONCRETE COLUMNS REINFORCED WITH STRUCTURAL SHAPES

BY

WILBUR LAWRENCE DUNN
GEORGE LEONARD JENSEN

THESIS

FOR THE

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

IN THE

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

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June 1, 1909

THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

WILBUR LAWRENCE DUNN and GEORGE LEONARD JENSEN

ENTITLED TESTS OF CONCRETE COLUMNS REINFORCED WITH STRUCTURAL SHAPES

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in Civil Engineering

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TESTS OF CONCRETE COLUMNS REINFORCED WITH STRUCTURAL SHAPES.

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PART I.

Introduction.

The series of tests described in this thesis was undertaken with the object of determining what effect may be expected upon the ultimate strength and deformation of reinforced concrete columns when the reinforcement is in the shape of angles, either laced or tied horizontally. Although many tests of columns with various percentages of reinforcement have been made but few have been on columns reinforced with structural shapes. The few tests of this kind have not resulted in definite conclusions as to the action of this type of reinforcement under stress.

It was expected that the tests made for this thesis would give valuable information bearing on, 1st- the effect of high percentages of reinforcement, and 2nd- the relative values of the two kinds of reinforcement described in this thesis, laced and tied angles. Comparisons will be made with tests of combined spiral and horizontal reinforcement to see if the structural shape reinforcement acts as band reinforcement as has been claimed by an engineer who will be quoted later on. The relative efficiency of the horizontally tied and the laced column will also be investigated.

Eight of the ten columns used in these tests are similar to those used in the construction of the McGraw Building of New York, designed by Professor W. H. Burr. The two other columns are similar to those used in the construction of the Watson Building of Chicago, designed by T. L. Condron and F. F. Sinks, Architects. In this connection a description of the conditions leading up to the design of the columns of this kind may be interesting.

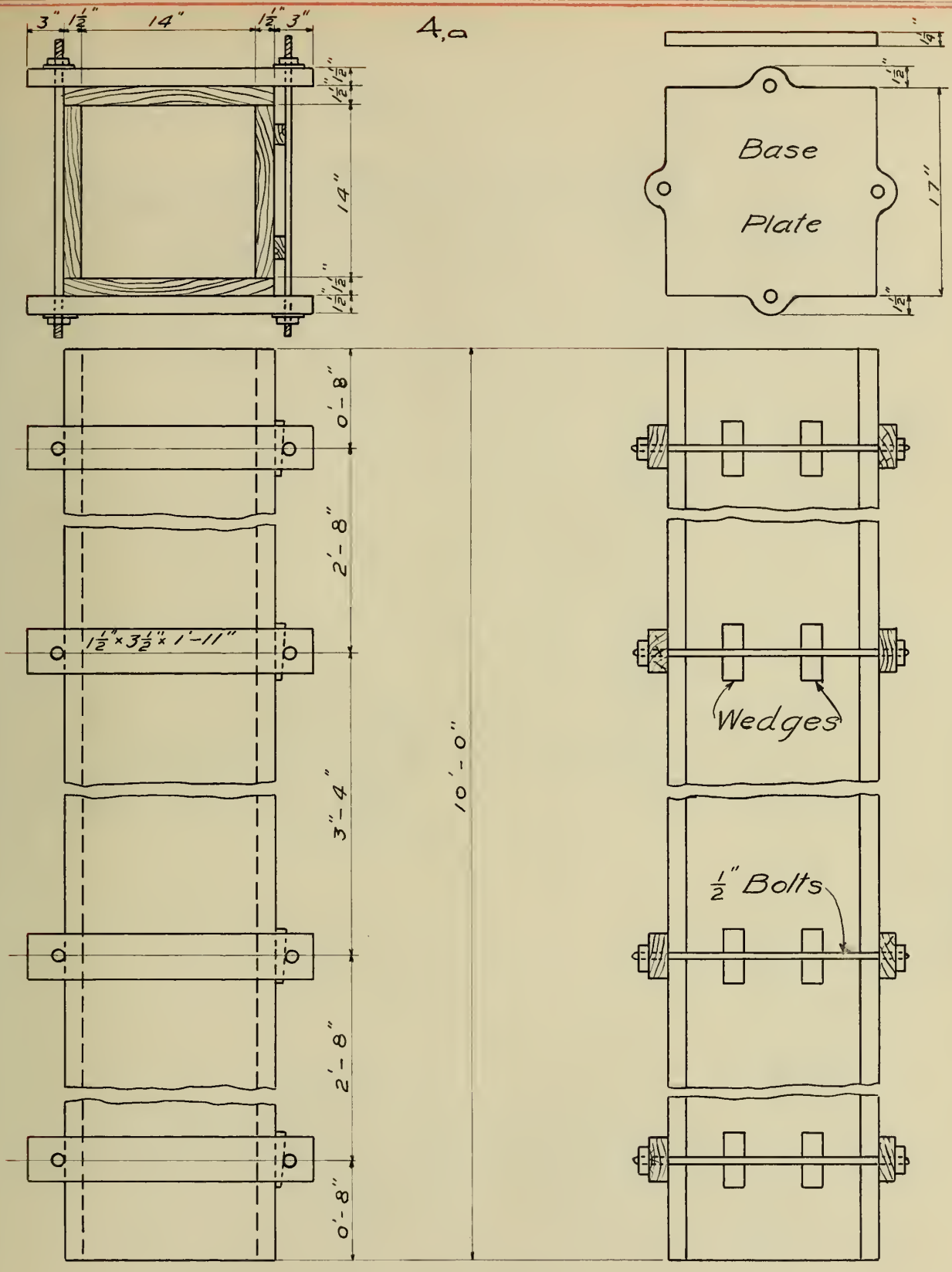
Up to the time of the design of the McGraw Building, the Building Code of New York permitted a working stress of only 350 pounds per square inch for concrete in direct compression it being practically assumed that all reinforcement would be of the Considere type. With this exceedingly low permissible stress the design of such a building, an eleven story structure subject to the severe vibratory stresses of a printing establishment, was an exceedingly uneconomical and altogether unsatisfactory proposition. The columns would have to be either of the ordinary size, spaced very close together or else of unusually large dimensions with ordinary spacing. Either of these conditions was undesirable in the McGraw Building where space for machinery and business purposes is valuable. Professor Burr, however, was certain that columns could be designed with reinforcement so placed as to justify larger working stress and consequently smaller columns. The column so designed is described roughly in the early part of this article and in detail in the Transactions of the American Society of Civil Engineers, Volume XXXIII. This column design was submitted to the

Bureau of Building of the City of New York for its examination and approval. The Bureau was favorably impressed with it and a special regulation was made permitting a working stress of 750 pounds per square inch for columns of this type, more than twice as much as for the ordinary column. This increased permissible load upon the concrete was coupled with the provision that the cross-section of the steel in any column at any floor should be sufficient to carry the entire dead load above that section without stressing the steel more than 16000 pounds per square inch.

Several statements made by Professor Burr in his paper before the American Society of Civil Engineers when speaking about the columns for this building will be discussed in this thesis. He said in part, " The use of the steel in load supporting conditions as a long column independent of concrete and at the same time forming a rigid banding member for the latter, with the consequent increase of permissible working load on the concrete, reduced the size of the columns------ with the desired convenient to dimensions consistent and economical use of clear floor space "-------." The lacing bars, rivet heads, and other projecting column details act positively in creating a firm and complete hold or bond between the steel-work of each column and the concrete enclosed within it. This feature of this column compels the steel and the enclosed concrete to act as a unit--------".

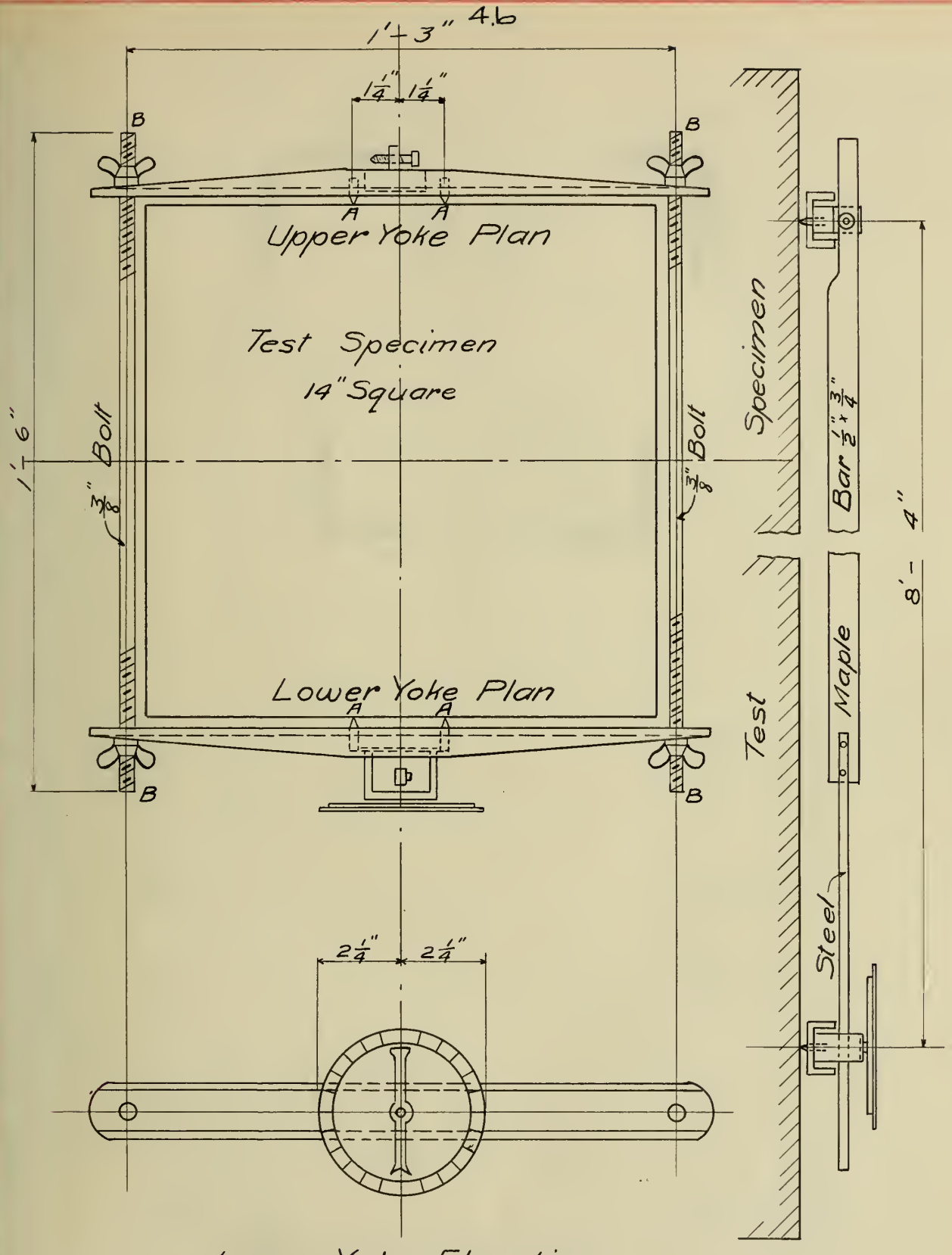
In the case of the columns designed by T. L. Condron and F. F. Sinks for the Watson Building in Chicago the angles were

horizontally tied instead of laced. In designing the columns the ratio of the moduli of elasticity of the steel and concrete was assumed as 15. The columns were not considered as hooped concrete only 500 pounds per square inch being allowed for the working stress of the concrete and 7500 pounds per square inch for the steel.



Form For Square Columns

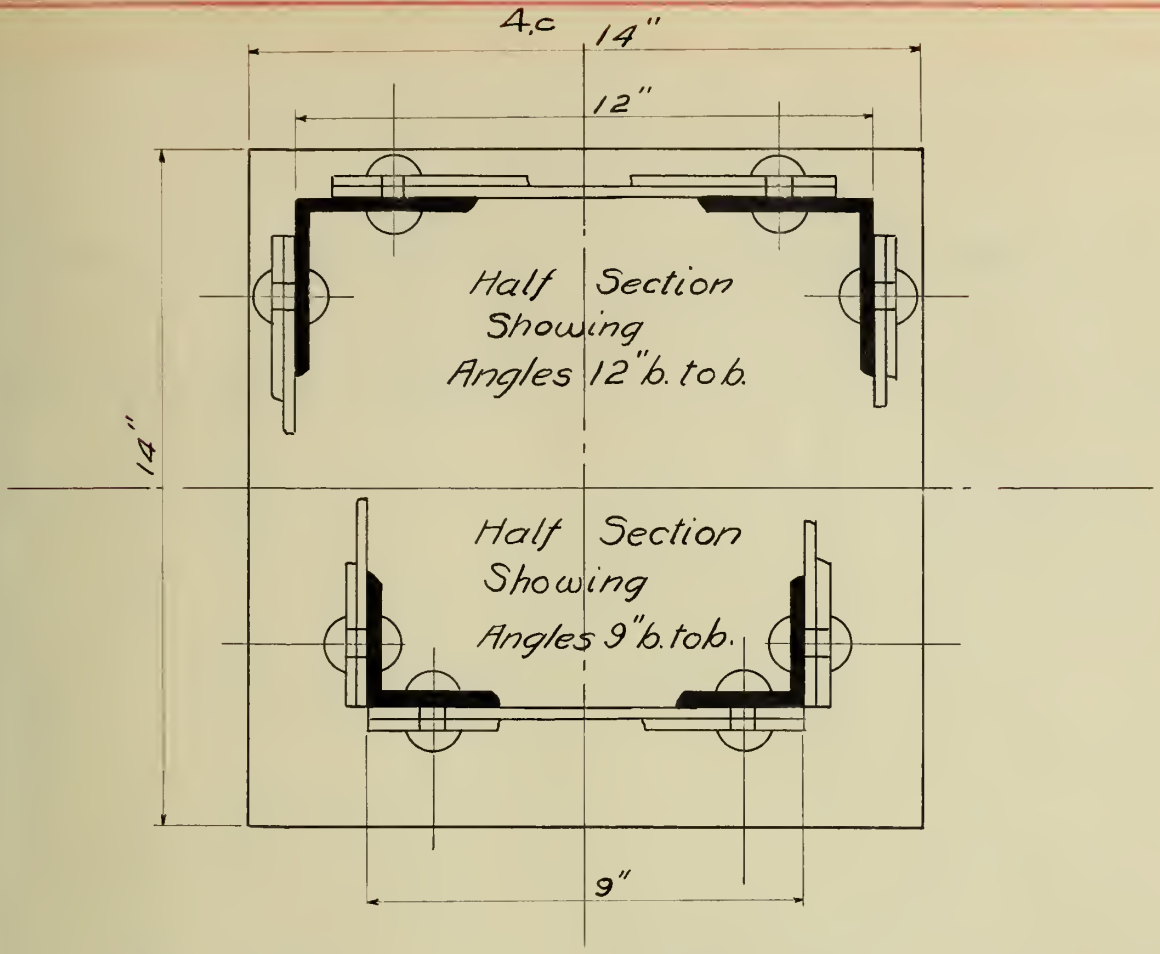
Fig. 1.

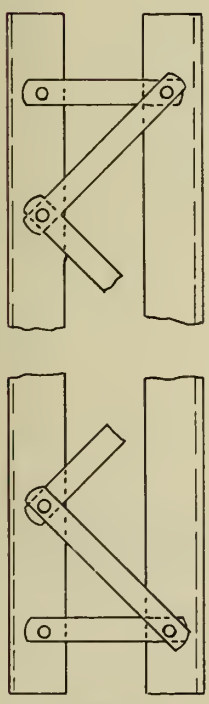


Lower Yoke Elevation

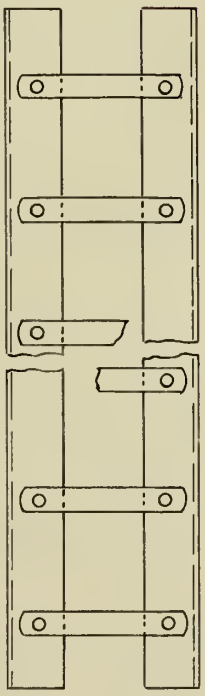
Longitudinal Extensometers
Used on Columns

Fig. 2.



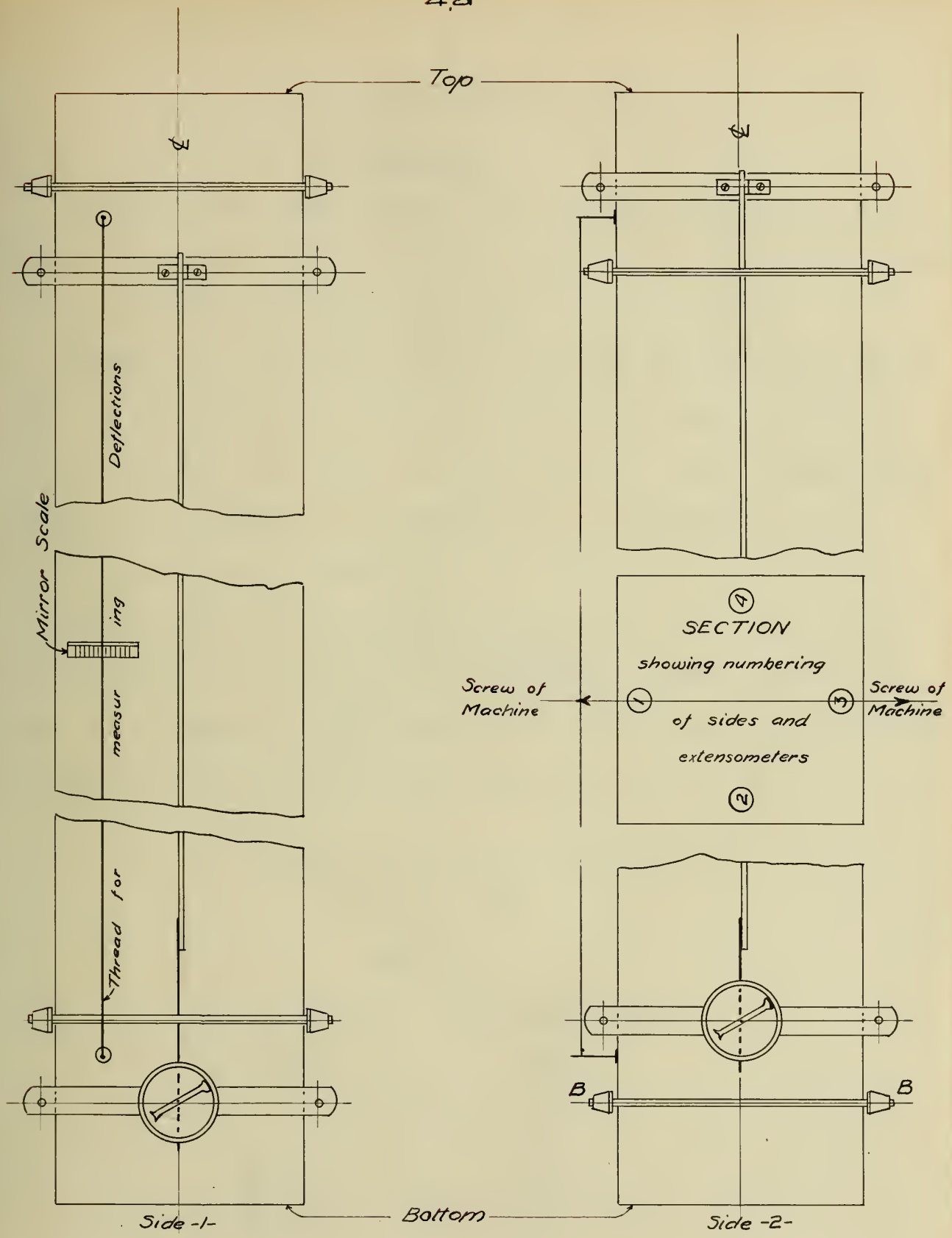


Method of
Diagonal Lacing



Method of
Horizontal Lacing

Fig. 3.



ARRANGEMENT OF APPARATUS

Fig. 4.

PART II.

Materials.

The materials used in making the columns were similar to the materials commonly used in concrete construction. The sand, stone and much of the cement were obtained in the open market.

1. STONE.- The stone was crushed limestone from Kankakee, Illinois. It was ordered to pass over a 1/4 inch screen and through a 1 inch screen. The stone for the different columns was slightly different in character as is shown in the accompanying tables. Columns 56, 61, and 63 were made with stone of the 1907 series; columns 31, 32, 34, 40, 45, and 48 with stone of the 1908 series, and column 52 with stone of the 1909 series. Table I gives the mechanical analysis of the stone of the 1907 series; Table II of the 1908 series, and Table III of the 1909 series.

Mechanical Analysis of Stone.

Series of 1907.

Table I.

Size of Mesh.	Percent Passing.
1 1/4 inch.	100
1 "	95
1/2 "	35
1/4 "	5
1/8 "	2

Contained about 50 % Voids.

Series of 1908.

Table II.

Size of Mesh.	Percent Passing.
1 inch.	100
3/4 "	93
1/2 "	64
3/8 "	41
1/3 "	22
1/5 "	4
1/10 "	2

Contained about 50 % Voids.

Series of 1909.

Table III.

Size of Mesh.	Percent Passing.
1 inch.	100
3/4 "	88
1/2 "	51
3/8 "	29
1/3 "	14
1/5 "	2
1/10 "	1

Three tests of this stone gave 50.0 , 45.8 and 49.7 % of Voids.

2. SAND.- The sand was of good quality, sharp, well-graded and fairly clean. It came from near the Wabash River at Attica, Indiana. A series of 18 tests gave the mean results given in Table 1V.

Table IV.

Mechanical Analysis of Sand.

Series of 1909.

Sieve No.	Separation Size Inches.	Percent Passing.
3		99.7
5	.174	95.9
10	.091	77.0
12	.067	69.8
16		62.9
18	.043	51.5
30	.027	32.5
40	.019	17.0
50	.013	5.8
74	.009	3.1
150		0.8

3. CEMENT.- In all 10 columns Universal Portland Cement was used. Tables V, VI, and VII give the results of tension, time of set, and fineness tests made in the University of Illinois Cement Laboratory.

Table V.

Time of Set.

	Gauged.	Initial Set.	Final Set.
	11:15	12:55	2:25
Mean.		1 hr. 40 min.	3 hr. 10 min.
	11:05	1:10	2:30
		2 hr. 5 min.	3 hr. 25 min.
Mean		1 hr. 52 min.	3 hr. 17 min.

Vicat Needle Test. Normal Consistency 22.5 % Water.

Table VI.

Tensile Strength.

No.	7 Day Test. Neat		7 Day Test. 1 : 3		28 Day Test. Neat		28 Day Test. 1 : 3	
	Trials 1st	2nd	1st	2nd	1st	2nd	1st	2nd
1	610	605	185	180	770	870	295	310
2	570	620	175	135	795	880	280	265
3	595	600	170	170	750	835	290	255
4	605	630	190	145	790	830	265	265
5	595	630	175	170	755	850	270	300
Mean	606		170		812		279	

Percent Water used ; Neat 22.0 With Sand 9.15

Kind of Sand Ottawa 20-30.

Table VII.

Fineness Test of Cement.

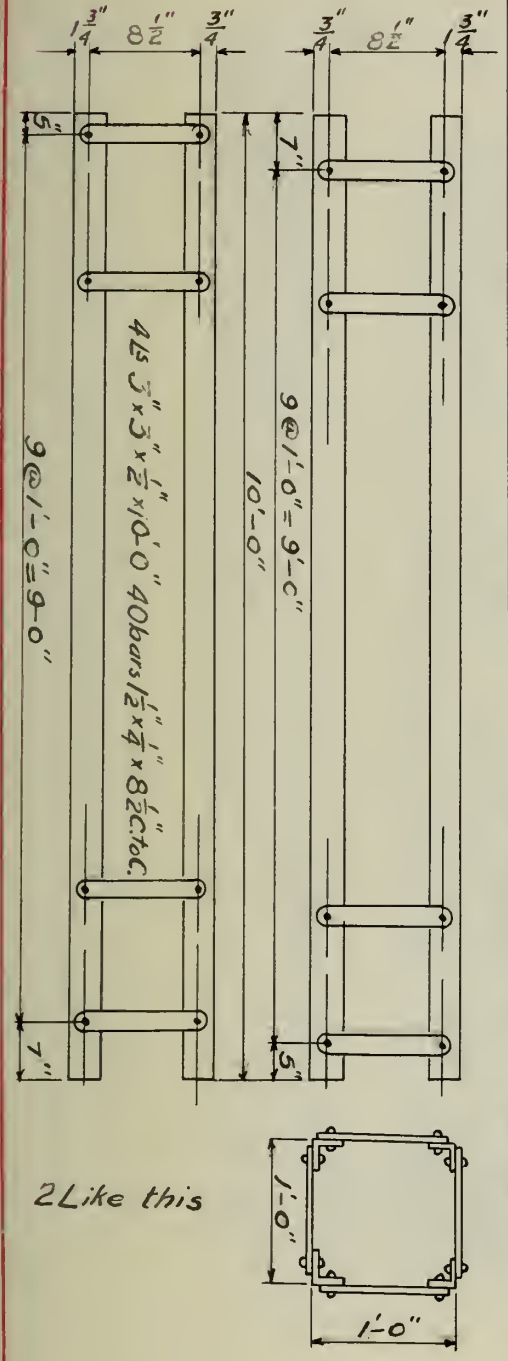
1000 Units Used.

Standard Mesh.	Quantity Retained.	Quantity Passing.	Percent Retained.	Percent Passing.
No. 50	12	988	1.2	98.8
No. 100	37	963	3.7	96.3
No. 200	187	813	18.7	81.3

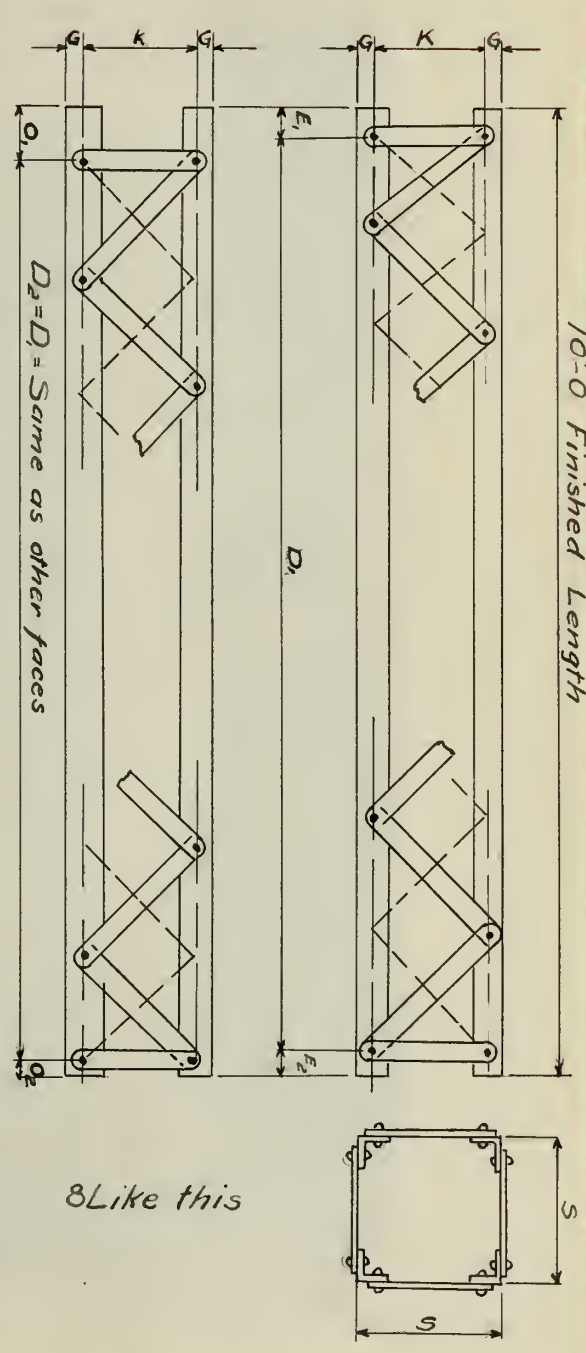
4. CONCRETE.- Men accustomed to mixing concrete made the test specimens and an effort was made to have the concrete as good as would be used in the best practice. All materials were proportioned by loose volume and weights were taken as a check on the measurements. The mixing was done with shovels by hand. The sand and cement were first mixed dry. The stone, which had previously been thoroughly moistened, was added and the mass turned until uniform in appearance. Water was added in such proportion as to give a fairly wet mixture. This consistancy permitted tamping into the forms to good advantage. All the concrete for these columns was made of a 1 - 2 - 4 mixture, proportioned by loose volume.

5. STEEL.- The reinforcement used in these columns consisted of angles either laced or horizontally tied. The angles were of the ordinary mild steel used in the standard structural shapes. The steel columns were fabricated by the Morava Construction Company, Engineers and Contractors, Chicago. The accompanying sketch shows

the steel reinforcement as fabricated by this company.

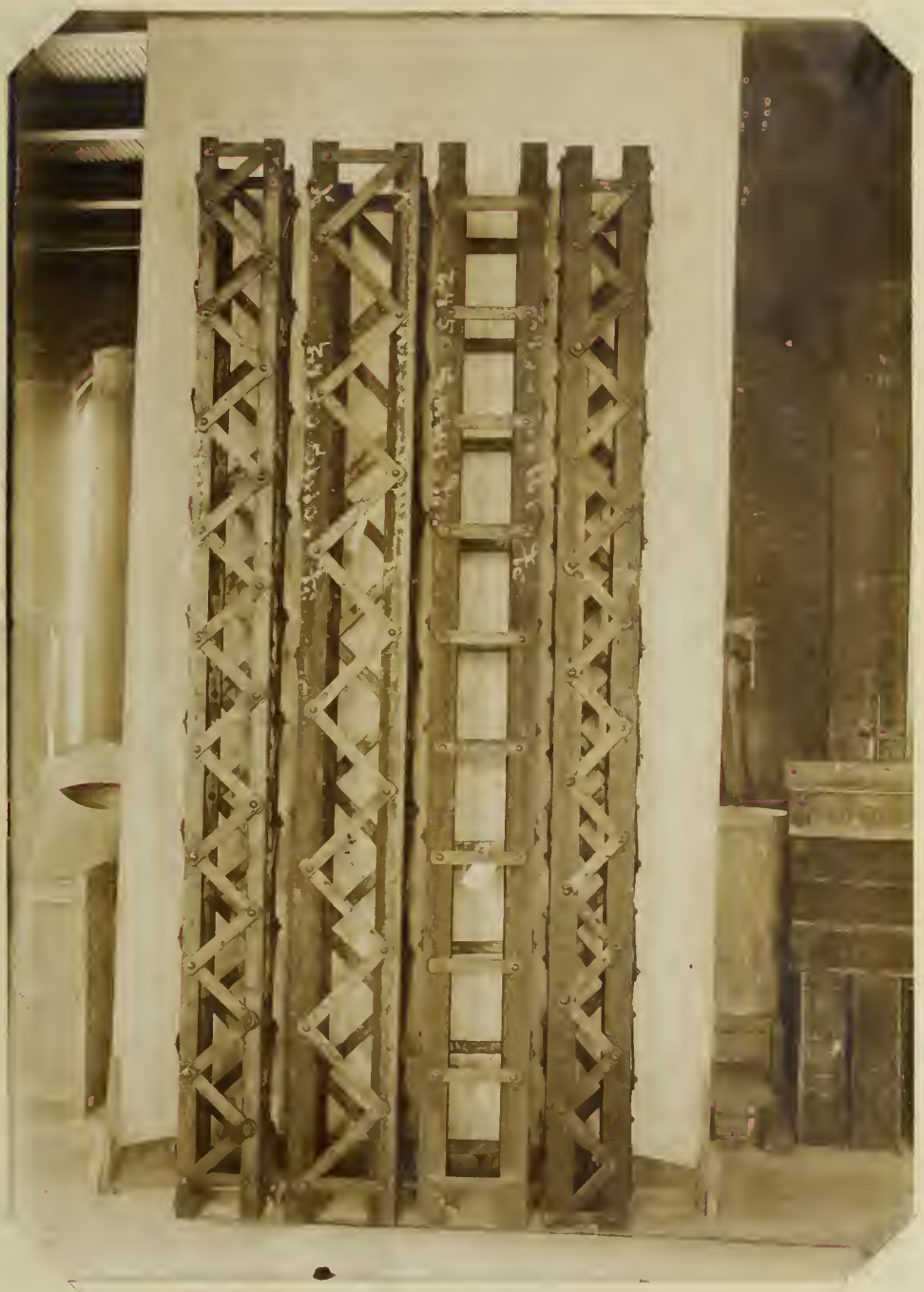


No. Pcs.	Material	S.	G.	K.	E_1	E_2	O_1	O_2	$D_1 = D_2$
2	8 ls 2"x2"x⅜"x10'-0"	9"	1¼"	6½"	1½"	4½"	4½"	1½"	19 @ 6" = 9'6"
2	8 ls 3"x3"x½" 1" x 10'-0"	9"	1¼"	5½"	1½"	4½"	4½"	1½"	do.
2	8 ls 3"x3"x½" 1" x 10'-0"	12"	1¼"	8½"	1½"	4½"	4½"	1½"	13 @ 8¾" = 9'-5¾"
2	8 ls 3"x3"x¼" x 10'-0"	do.	1¼"	8½"	1½"	4½"	4½"	1½"	do.



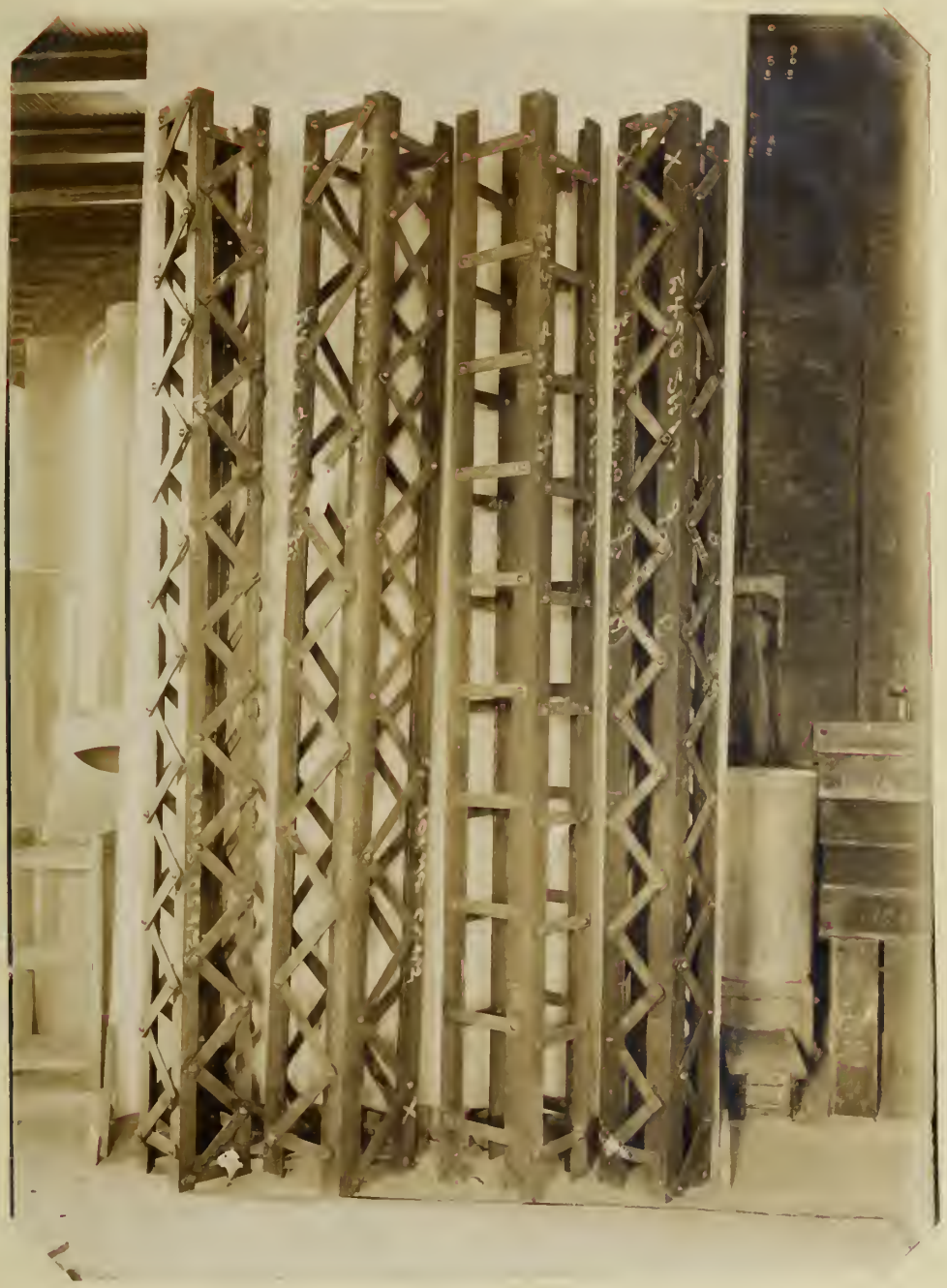
All rivets ½"
No Paint

Fig. 5.



View showing the 4 styles of reinforcement used.

Fig. 5a.



View showing the 4 styles of reinforcement used.

Fig. 5b.

PART III.

Test Specimens.

In making the test specimens every effort was made to have the conditions under which they were made as uniform as possible. Two columns of each kind of reinforcement were made. This was done to insure more consistent results. One column would act as a check on the other and there would thus be less danger of basing conclusions upon tests which may have been made on columns of inferior quality. ~~Two~~ Three cubes were usually made for each column.

1. CUBES.- The cubes were all six inches on an edge. They were made of the same concrete as the correspondingly numbered columns.

The concrete for the cubes was placed in the cube forms at the same time that the concrete was placed in the column forms. The conditions of storage were practically the same for the cubes as for the columns, the forms were taken from the cubes the same time that they were taken from the columns, and in every way possible the concrete in the cubes was made and treated the same as that in the columns.

2. Columns. - 10 Columns were made . The type of reinforcement used is shown in photographs 5a, 5b, and in Fig. 3 All columns were made square in section of about 14 inches by 14 inches in dimensions. All columns were made 10 feet long.

a. Forms.- The forms for the columns were made of 2 inch pine plank, planed on both sides. Drawings of the forms are shown in Fig. 1. Each of the four sides of the forms was made

of two continuous pieces cleated transversely and extending the full length of the column. The forms were held together with ties of 2 inch by 4 inch pieces and 1/2 inch bolts. As shown in Fig. 1. four of these braces were used, one at each end and two intermediately. Wedges were used to adjust the form to the correct width and to hold it while the concrete was being put in .

b. Making of Columns.- The forms were soaked in water for a few days before being set up, this kept the wood from drawing very much water out of the concrete and also decreased the tendency of the form to warp.

The reinforcement was put in place and rested squarely upon the cast iron base plate. The angles of the reinforcement had been planed after the reinforcement had been assembled so as to insure that this would be the case. The wooden form was then centered about the reinforcement. The concrete was mixed in one batch. After thoroughly mixing, as previously described in this thesis, the concrete was put into the forms in 6 inch layers, each layer being thoroughly tamped or churned, troweled around the sides to improve the surface, and then tamped again. The consistency of the mixture was such that with a 4 inch 10 pound tamper, efforts at tamping generally resulted in churning the mass, and water was constantly present at the top. The column was thus built up in 6 inch sections, the concrete being placed in the column by means of buckets, each bucket-full constituting one of these sections. The forms were filled up in this way to the full height of the

steel reinforcement and then an additional inch or two of concrete was placed on top of the steel reinforcement so that the bearing of the machine head would not be upon the steel. After testing columns 31, 32, and 34, however, the top of the remaining columns were cut down to about 1/2 inch below the ends of the angles. Each column was then built up with neat cement mortar flush with the ends of the reinforcing angles. The cast iron bearing plate was placed in position upon the ends of the steel angles while the cement was still plastic. About a week was given the cement to set in before testing. It was now possible to secure a uniform distribution of stress to the concrete and steel.

c, STORAGE OF COLUMNS.- The columns were built near the walls of the testing room of the Laboratory of Applied Mechanics and remained there until the time of the test. The forms were taken off about 10 days after making the columns. The temperature of the room varied from 55° to 70° F. The 6 inch cubes were stored in damp sand until a day or two previous to being tested.

PART IV.

Detail of Test.

1. TESTING MACHINE USED.- The machine used in testing the columns was the 600 000 pound Riehle Vertical Screw Machine of the Laboratory of Applied Mechanics. The slowest speed, 0.05 inches per minute, was used on all the tests. The 6 inch cubes were tested in an Olsen Testing Machine of 100 000 or 200 000 pounds capacity.

2. INSTRUMENTS USED.- The instruments used for measuring the shortening of the columns were four Johnson Extensometers placed on the four faces of the column. The extensometers were so arranged as to read the deformation in the center of the face upon which they were placed. They were arranged in pairs, each pair of extensometers being mounted on the same yoke, Fig. 4 . The yokes were fastened to the column by means of through bolts, B, and a firm contact was insured by means of two lugs, A, on each arm. The lugs were of the dimensions shown in Fig. 2 , and were placed symmetrically with regard to the center lines of the column. The two yokes carrying the dials were placed 3 or 5 inches apart at the bottom of the column and the corresponding yokes carrying the extensometer bars were placed similarly at the top. The gauge length of 100 inches was set off centrally upon the length of the column and the instruments were set to this length. The extensometer bars were made of seasoned maple with steel blades attached to the lower ends. These blades were brought into good contact with the rollers of the dial by means of rubber bands which were stretched between them and the straps placed on the dial standards.

The dials, which were 4 1/2 inches in diameter, read to .001 of an inch on the dial graduations with a vernier attachment on the pointer reading to .0001 of an inch. The verniers, however, were not used very much because sufficiently close contact between vernier and scale could not be attained and also because the load readings were not exact enough to justify an accuracy in deformation reading of .0001 of an inch. One full revolution of the pointer around the dial gave a deformation of one half an inch.

3, GENERAL METHOD OF TESTING.- Before the columns were placed in the machine a cast iron plate similar to the base plate was set in plaster of paris on the top of the column. This plate acted as a bearing plate for the bearing block of the machine head after the column had been placed in the machine. Between the top and bottom bearing plates two rods were passed, which supported the columns while being carried to the machine. The columns were lifted from the floor by means of a travelling screw lift mounted on an overhead crane. Both the crane itself and the screw lift mounted upon it could be moved backward or forward, so that it was a comparatively simple task with this apparatus to place the columns upon the machine bed and to center it under the head of the machine. After centering the column the movable head of the machine was brought down upon the column. To avoid eccentricity of loading the pressure was transmitted from the machine head to the bearing plate through a spherical bearing block of the ball and socket type. At an initial load of six or seven thousand pounds all the extensometers were set at zero after which the load

was applied in increments of about 25 000 pounds. At each increment all the extensometers were read, two observers stationed on opposite sides of the column each reading two extensometers.

The lateral deflections of some of the columns were measured roughly by means of a thread drawn taut between two pin plates fastened to the column near the top and bottom bearing plates, and a double mirror fastened to the column at its mid point. These readings were used only as a check on the way the column was deflecting. See Fig. /8 .

The cubes were tested in the 100 000 or 200 000 pound Olson Machine as previously stated. No attempt at reading the deformation of the cubes was made. The load was applied until failure occurred the ultimate load being the only load noted.

PART V.

-----O-----

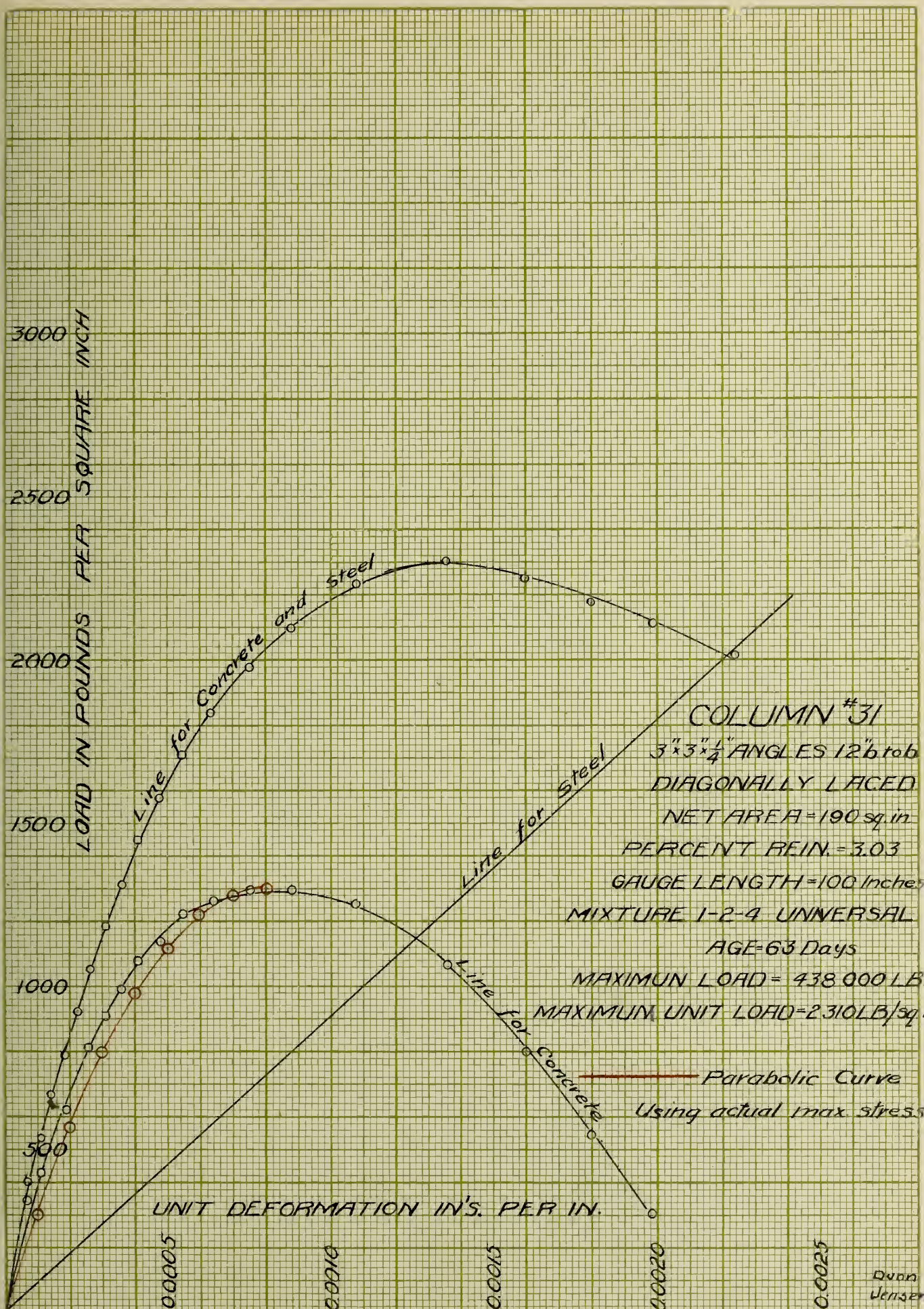
Observed and Computed Data.

COLUMN 31.

Observed and Computed Data.

Length, 9 ft. 11 1/2 in. — Mixture, 1-2-4.
Gauge Length, 100 in. — Age when tested, 63 days.
Cross-section 14 x 14 1/2 in. — Cement, Universal Portland.
Size of Angles, 3 x 3 x 1/4 in. — Distance Back to Back, 12 inches

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in.	per 1 in.
6700	36	.0000	.00000
26000	137	.0021	.00002
51000	268	.0048	.00005
76000	400	.0082	.00008
101000	531	.0121	.00012
126000	663	.0150	.00015
150000	790	.0187	.00019
175000	921	.0225	.00023
200000	1052	.0267	.00027
225000	1185	.0311	.00031
250000	1315	.0356	.00036
275000	1448	.0403	.00040
300000	1578	.0464	.00046
325000	1710	.0536	.00054
350000	1840	.0614	.00061
375000	1975	.0725	.00073
400000	2100	.0851	.00085
425000	2240	.1040	.00104
438000	2310	.1301	.00130
428000	2250	.1498	.00150
416000	2190	.1663	.00166
403000	2120	.1808	.00181
384000	2020	.2090	.00209



EUGENE DIETZGEN CO., CHICAGO.

Fig. 6.

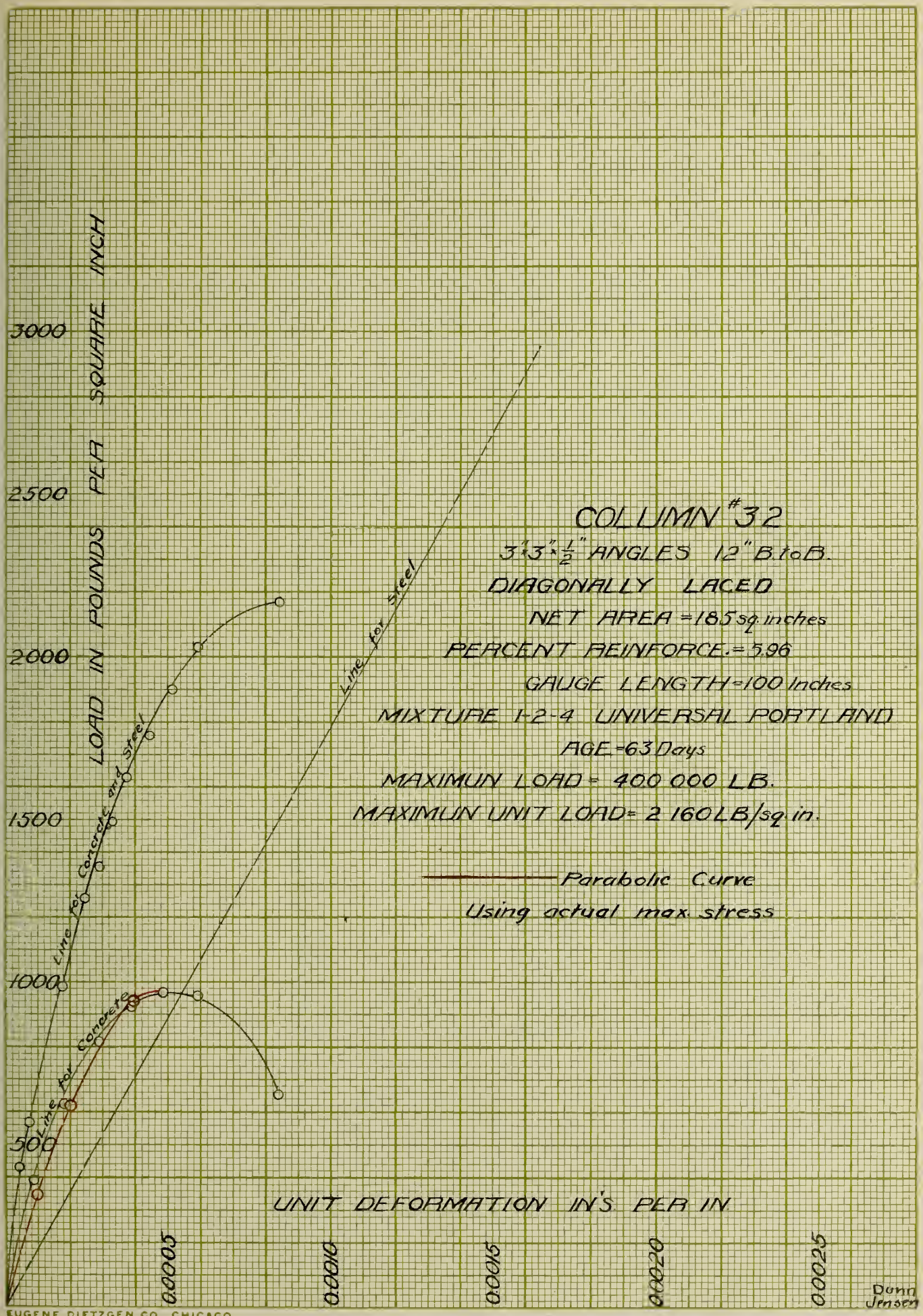
COLUMN 32.

Observed and Computed Data.

Length, 10 ft. 1 in.
Gauge Length, 100 in.
Cross-section, 14 x 14 1/8 in.
Size of Angles, 3 x 3 x 1/2 in.

Mixture 1- 2 - 4.
Age when tested, 63 days.
Cement, Universal.
Distance Back to Back, 12 in.

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in.	per 1 in.
6500	35	.0000	.00000
26000	141	.0013	.000013
51000	273	.0038	.00004
77000	416	.0062	.00006
101000	546	.0092	.00009
127000	686	.0126	.00013
151000	816	.0155	.00016
175000	945	.0185	.00019
201000	1085	.0221	.00022
225000	1215	.0257	.00026
250000	1350	.0295	.00030
275000	1485	.0340	.00034
300000	1620	.0390	.00039
325000	1755	.0450	.00045
350000	1890	.0526	.00053
375000	2020	.0600	.00060
400000	2160	.0854	.00085



EUGENE DIETZGEN CO., CHICAGO.

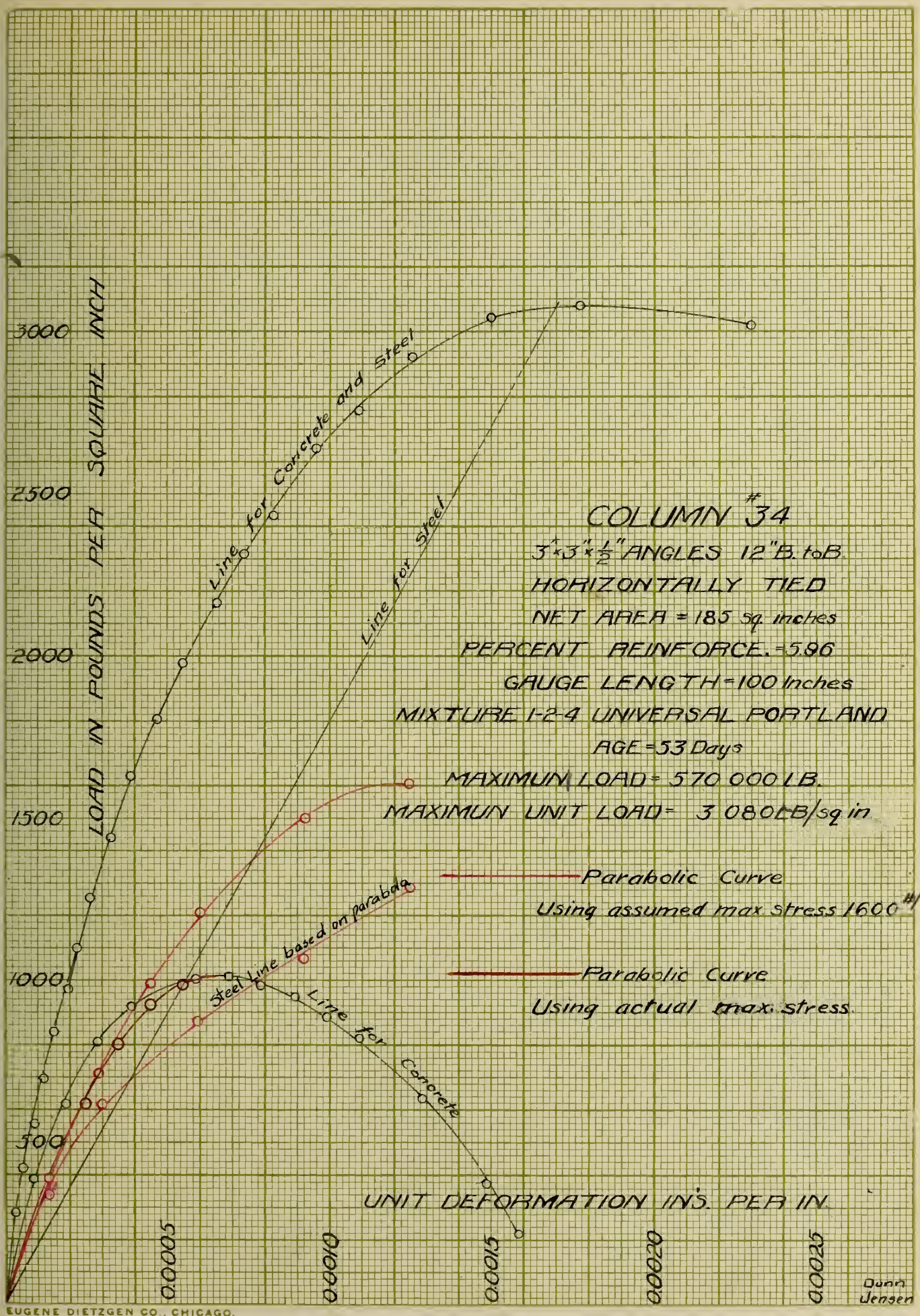
Fig. 7.

COLUMN 34.

Observed and Computed Data.

Length, 10 ft. 0 in.
Gauge Length, 100 in.
Cross-section, 13 3/4 x 14 in.
Size of Angles, 3 x 3 x 1/2 in.
Mixture, 1 - 2 - 4.
Age when tested, 53 days.
Cement, Universal Portland.
Distance Back to Back, 12 in.

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in.	per 1 in.
6500	35	.0000	.00000
26100	141	.0015	.00002
51000	276	.0040	.00004
75000	405	.0067	.00007
101000	545	.0100	.00010
126000	680	.0131	.00013
152000	820	.0164	.00016
177000	960	.0195	.00020
201000	1085	.0226	.00023
230000	1240	.0265	.00027
266000	1435	.0337	.00034
300000	1620	.0393	.00039
333000	1800	.0475	.00048
364000	1970	.0548	.00055
400000	2160	.0656	.00066
427000	2310	.0738	.00074
450000	2430	.0828	.00083
488000	2640	.0954	.00095
509000	2750	.1087	.00109
540000	2920	.1252	.00125
564000	3050	.1498	.00150
570000	3080	.1774	.00177
560000	3030	.2300	.00230



EUGENE DIETZGEN CO., CHICAGO.

Fig. 8

COLUMN 40.

Observed and Computed Data.

Length, 10 ft. 0 in. Mixture, 1 - 2 - 4.
Gauge Length, 100 in. Age when tested, 64 days.
Cross-section, 14 1/2 x 13 1/2 in. Cement, Universal Portland.
Size of Angles, 2 x 2 x 3/8 in. Distance Back to Back, 9 in.

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in.	per 1 in.
7000	38	.0000	.00000
26000	136	.0034	.00003
51000	276	.0075	.00008
76000	398	.0116	.00012
101000	540	.0163	.00016
126000	660	.0209	.00021
150000	786	.0256	.00026
175000	920	.0294	.00029
201000	1050	.0362	.00036
226000	1185	.0418	.00042
251000	1315	.0479	.00048
275000	1440	.0548	.00055
300000	1570	.0619	.00062
325000	1700	.0707	.00071
375000	1960	.0914	.00091
400000	2090	.1048	.00105
419000	2200	.1438	.00144
398000	2080	.1680	.00168
356000	1865	.1871	.00187

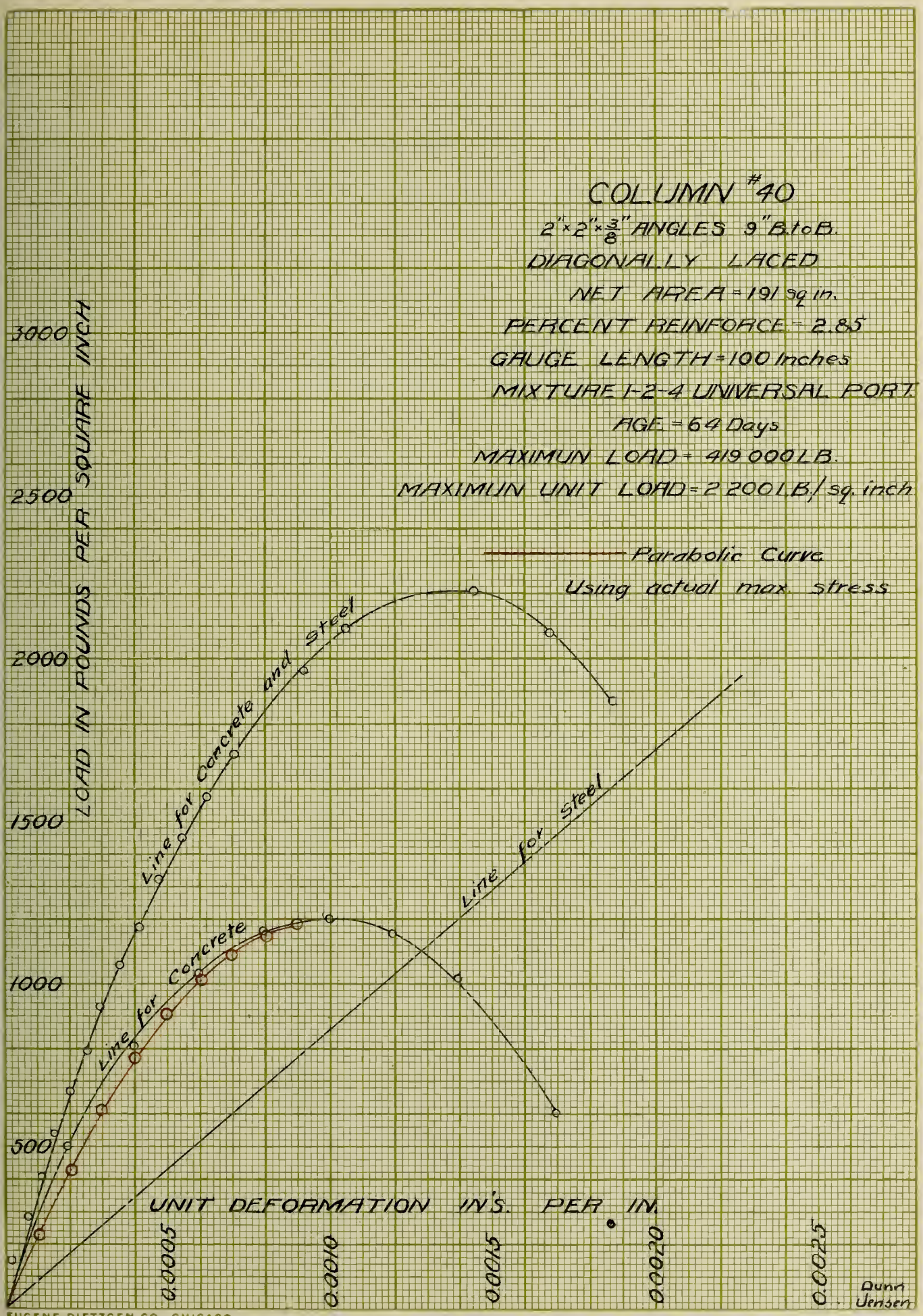

COLUMN #40
2"×2"×3/8" ANGLES 9" B.to B.
DIAGONALLY LACED
NET AREA = 191 sq in.
PERCENT REINFORCE = 2.85
GAUGE LENGTH = 100 Inches
MIXTURE 1-2-4 UNIVERSAL PORT.
AGE = 64 Days
MAXIMUN LOAD = 419 000 LB.
MAXIMUN UNIT LOAD = 2 200 LB./sq. inch
Parabolic Curve
Using actual max. stress
LOAD IN POUNDS PER SQUARE INCH
3000
2500
2000
1500
1000
500
Line for Concrete and Steel
Line for Concrete
Line for Steel
UNIT DEFORMATION IN'S. PER IN.
0.0005
0.0010
0.0015
0.0020
0.0025
Dunn Jensen
EUGENE DIETZGEN CO., CHICAGO.

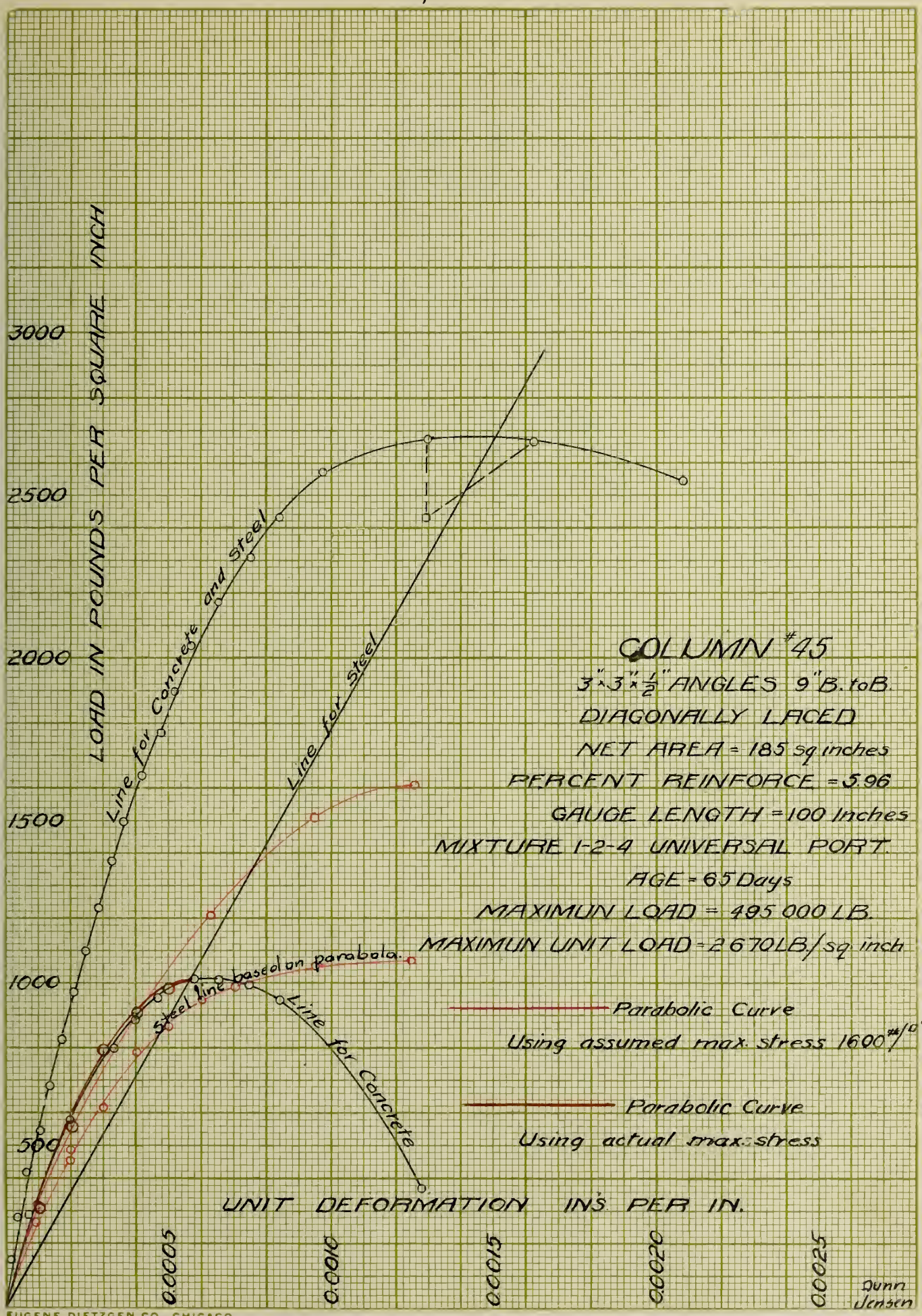
Fig. 9.

COLUMN 45.

Observed and Computed Data.

Length, 10 ft. 0 in.
Gauge Length, 100 in.
Cross-section, 13 3/4 x 14 1/2 in.
Size os Angles, 3 x 3 x 1/2 in.
Mixture, 1 - 2 - 4.
Age when tested, 65 days.
Cement, Universal Portland.
Distance Back to Back, 9 in.

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in	per 1 in.
8200	44	.0000	.00000
25000	135	.0018	.00002
50000	270	.0042	.00004
75000	405	.0072	.00007
99000	535	.0112	.00011
125000	675	.0139	.00014
150000	810	.0175	.00018
177000	960	.0211	.00021
202000	1090	.0248	.00025
226000	1220	.0289	.00029
253000	1370	.0328	.00033
275000	1490	.0370	.00037
302000	1630	.0415	.00042
327000	1770	.0475	.00048
350000	1890	.0524	.00052
375000	2030	.0573	.00057
401000	2170	.0647	.00065
425000	2300	.0746	.00075
450000	2430	.0839	.00084
475000	2570	.0968	.00097
495000	2670	.1291	.00129
495000	2670	.1615	.00162
475000	2570	.2078	.00208
425000	2300	.3013	.00301
400000	2160	.3917	.00392
394000	2130	.4467	.00447
392000	2120	.4627	.00463



EUGENE DIETZGEN CO., CHICAGO.

Fig. 10.

COLUMN 48.

Observed and Computed Data.

Length, 10 ft. 0 in.
Gauge Length, 100 in.
Cross-section, 14 x 14 in.
Size of Angles, 3 x 3 x 1/4 in.

Mixture 1 - 2 - 4.
Age when tested, 65 days.
Cement, Universal Portland.
Distance, Back to Back, 12 in.

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in.	per 1 in.
6600	35	.0000	.00000
25000	131	.0023	.00002
50000	262	.0054	.00005
76000	400	.0090	.00009
100000	525	.0127	.00013
127000	670	.0164	.00016
170000	895	.0240	.00024
201000	1060	.0295	.00030
226000	1190	.0344	.00034
251000	1320	.0394	.00039
276000	1450	.0449	.00045
301000	1590	.0509	.00051
326000	1720	.0587	.00059
351000	1850	.0665	.00067
375000	1970	.0769	.00077
401000	2110	.0901	.00090
426000	2440	.1098	.00110
440000	2310	.1351	.00135
428000	2250	.1614	.00161
403000	2120	.1913	.00191
375000	1970	.2126	.00213
50000	1840	.2252	.00225

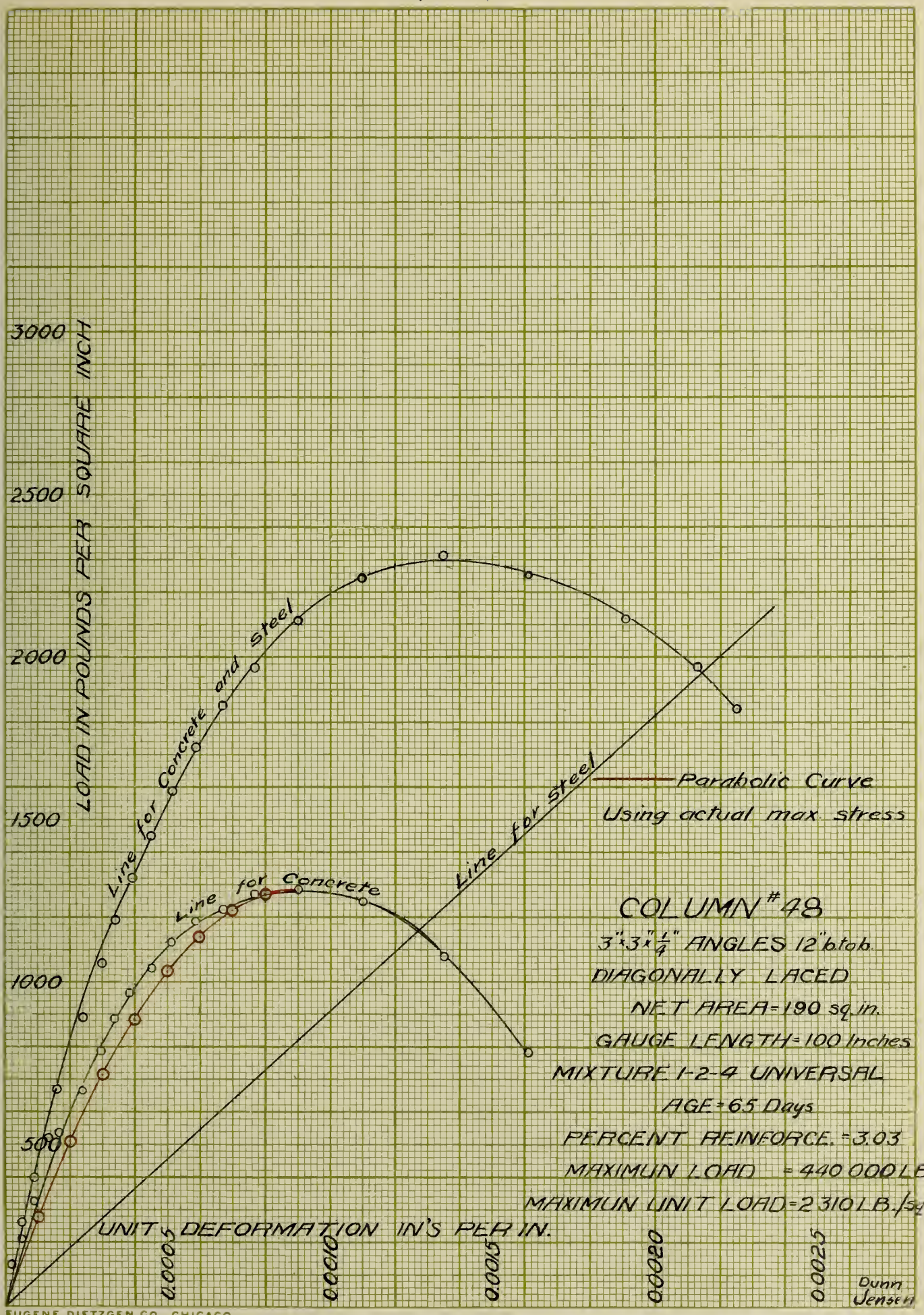

LOAD IN POUNDS PER SQUARE INCH
3000
2500
2000
1500
1000
500
Line for Concrete and Steel
Line for Concrete
Line for Steel
Parabolic Curve
Using actual max. stress
COLUMN # 48
3"x3"x1/4" ANGLES 12" b.to.b.
DIAGONALLY LACED
NET AREA = 190 sq. in.
GAUGE LENGTH = 100 Inches
MIXTURE 1-2-4 UNIVERSAL
AGE = 65 Days
PERCENT REINFORCE. = 3.03
MAXIMUN LOAD = 440 000 LB
MAXIMUN UNIT LOAD = 2310 LB./sq
UNIT DEFORMATION IN'S PER IN.
0.0005
0.0010
0.0015
0.0020
0.0025
Dunn
Jensen
EUGENE DIETZGEN CO., CHICAGO.

Fig. 11.

COLUMN 52.

Observed and Computed Data.

Length, 10 ft. 0 in.
Gauge Length, 100 in.
Cross-section, 14 x 14 in.
Size of Angles, 3 x 3 x 1/2 in.
Mixture, 1 - 2 - 4.
Age when tested, 61 days.
Cement, Universal Portland.
Distance, Back to Back, 12 in.

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in.	per 1 in.
6700	36	.0000	.00000
31000	168	.0026	.00003
60000	324	.0070	.00007
92000	497	.0113	.00011
125000	677	.0159	.00016
152000	822	.0201	.00020
182000	985	.0263	.00026
211000	1140	.0332	.00033
242000	1310	.0413	.00041
272000	1470	.0448	.00045
301000	1625	.0524	.00052
326000	1760	.0592	.00059
361000	1950	.0700	.00070
381000	2060	.0786	.00079
401000	2160	.0863	.00086
425000	2290	..1012	.00101
444000	2400	.1370	.00137
438000	2370	.1890	.00190
420000	2270	.2438	.00244

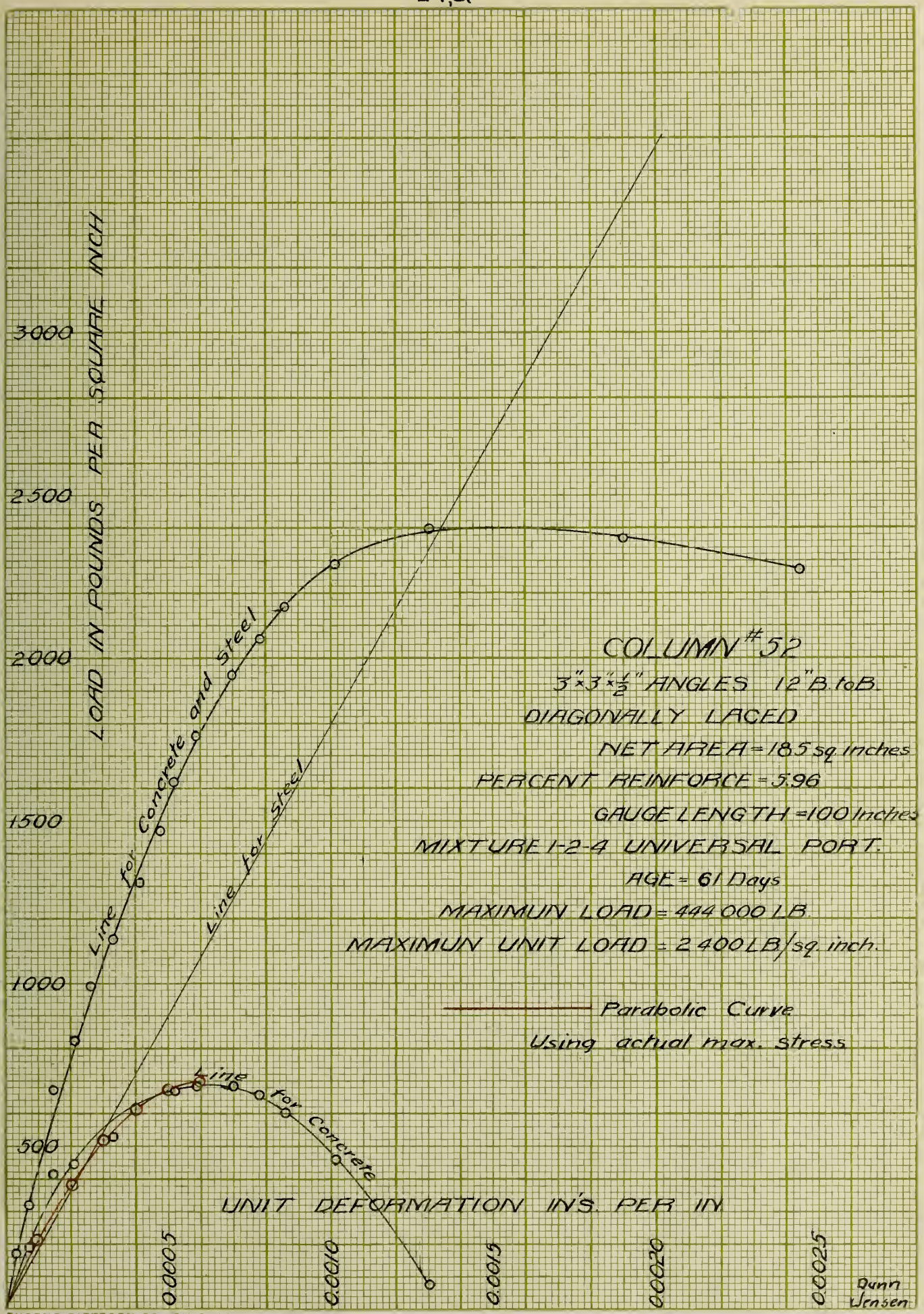

LOAD IN POUNDS PER SQUARE INCH
3000
2500
2000
1500
1000
500
Line for Concrete and Steel
Line for Steel
Line for Concrete
COLUMN # 52
3" × 3" × 1/2" ANGLES 12" B. to B.
DIAGONALLY LACED
NET AREA = 18.5 sq. inches
PERCENT REINFORCE = 5.96
GAUGE LENGTH = 100 Inches
MIXTURE 1-2-4 UNIVERSAL PORT.
AGE = 61 Days
MAXIMUN LOAD = 444 000 LB.
MAXIMUN UNIT LOAD = 2 400 LB./sq. inch.
Parabolic Curve
Using actual max. stress
UNIT DEFORMATION IN'S. PER IN.
0.0005
0.0010
0.0015
0.0020
0.0025
Dunn
Jensen
EUGENE DIETZGEN CO., CHICAGO.

Fig. 12.

COLUMN 56.

Observed and Computed Data.

Length, 10 ft. 0 in.
Gauge Length 100 in.
Cross-section, 14 x 14 in.
Size of Angles, 3 x 3 x 1/2 in.

Mixture 1 - 2 - 4.
Age when tested, 59 days.
Cement, Universal Portland.
Distance, Back to Back, 12 in.

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in.	per 1 in.
7000	38	.0000	.00000
26000	141	.0029	.00003
53000	287	.0065	.00007
76000	411	.0110	.00011
100000	542	.0160	.00016
127000	687	.0212	.00021
150000	812	.0253	.00025
176000	953	.0313	.00031
202000	1090	.0367	.00037
225000	1215	.0424	.00042
251000	1355	.0481	.00048
275000	1485	.0547	.00055
300000	1620	.0613	.00061
326000	1765	.0688	.00069
352000	1900	.0760	.00076
376000	2030	.0829	.00083
401000	2170	.0945	.00095
426000	2300	.1069	.00107
450000	2430	.1269	.00127
464000	2510	.1860	.00186
469000	2540	.2435	.00244
480000	2600	.3124	..00312
480000	2600	.3400	.00340
478000	2590	.3681	.00368

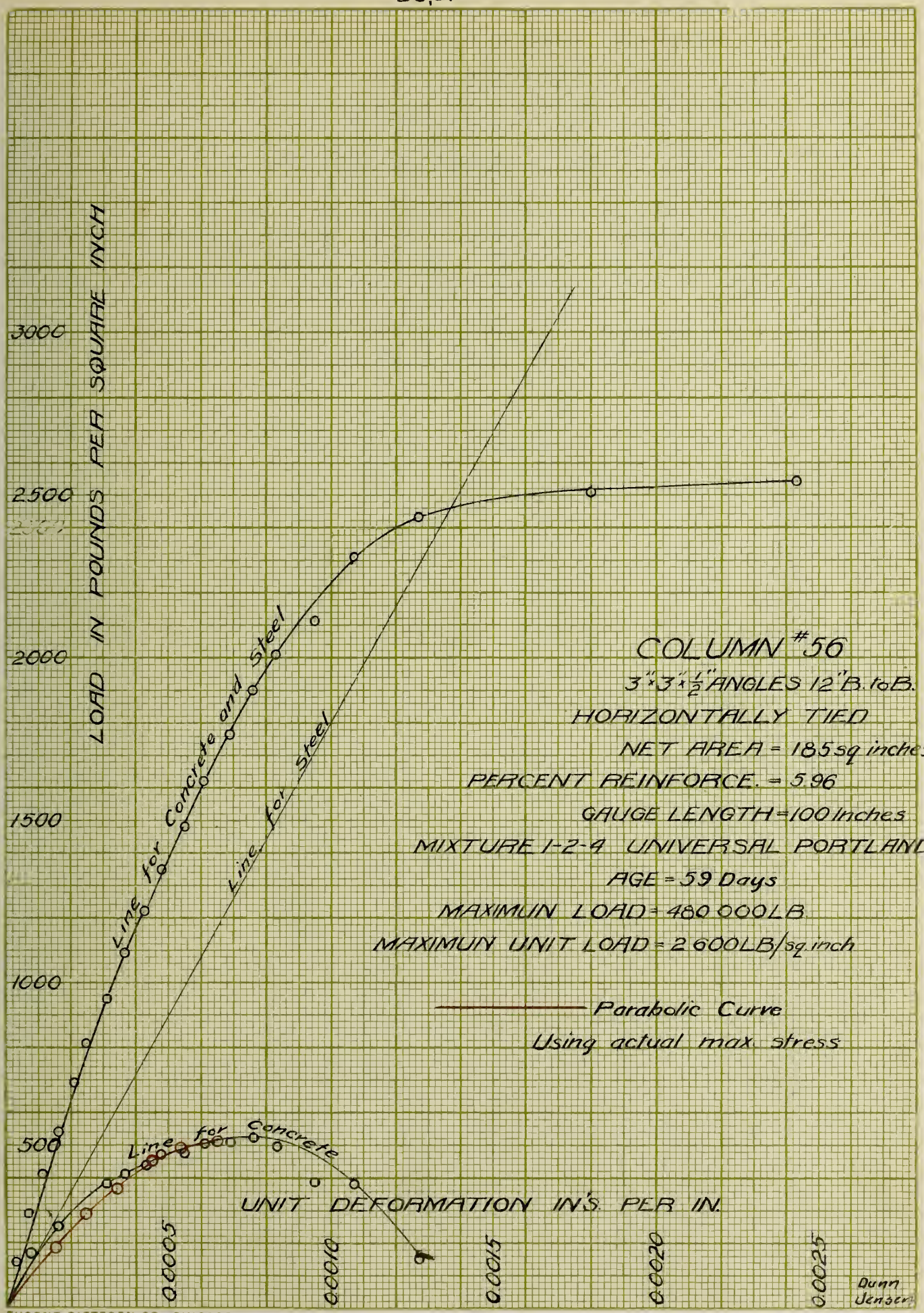

LOAD IN POUNDS PER SQUARE INCH
3000
2500
2000
1500
1000
500
Line for Concrete and Steel
Line for Steel
Line for Concrete
COLUMN #56
3"×3"×½" ANGLES 12" B. to B.
HORIZONTALLY TIED
NET AREA = 1.85 sq inches
PERCENT REINFORCE. = 5.96
GAUGE LENGTH = 100 Inches
MIXTURE 1-2-4 UNIVERSAL PORTLAND
AGE = 59 Days
MAXIMUN LOAD = 480 000 LB.
MAXIMUN UNIT LOAD = 2 600 LB/sq. inch
Parabolic Curve
Using actual max. stress
UNIT DEFORMATION IN'S. PER IN.
0.0005
0.0010
0.0015
0.0020
0.0025
Dunn
Jensen
EUGENE DIETZGEN CO., CHICAGO.

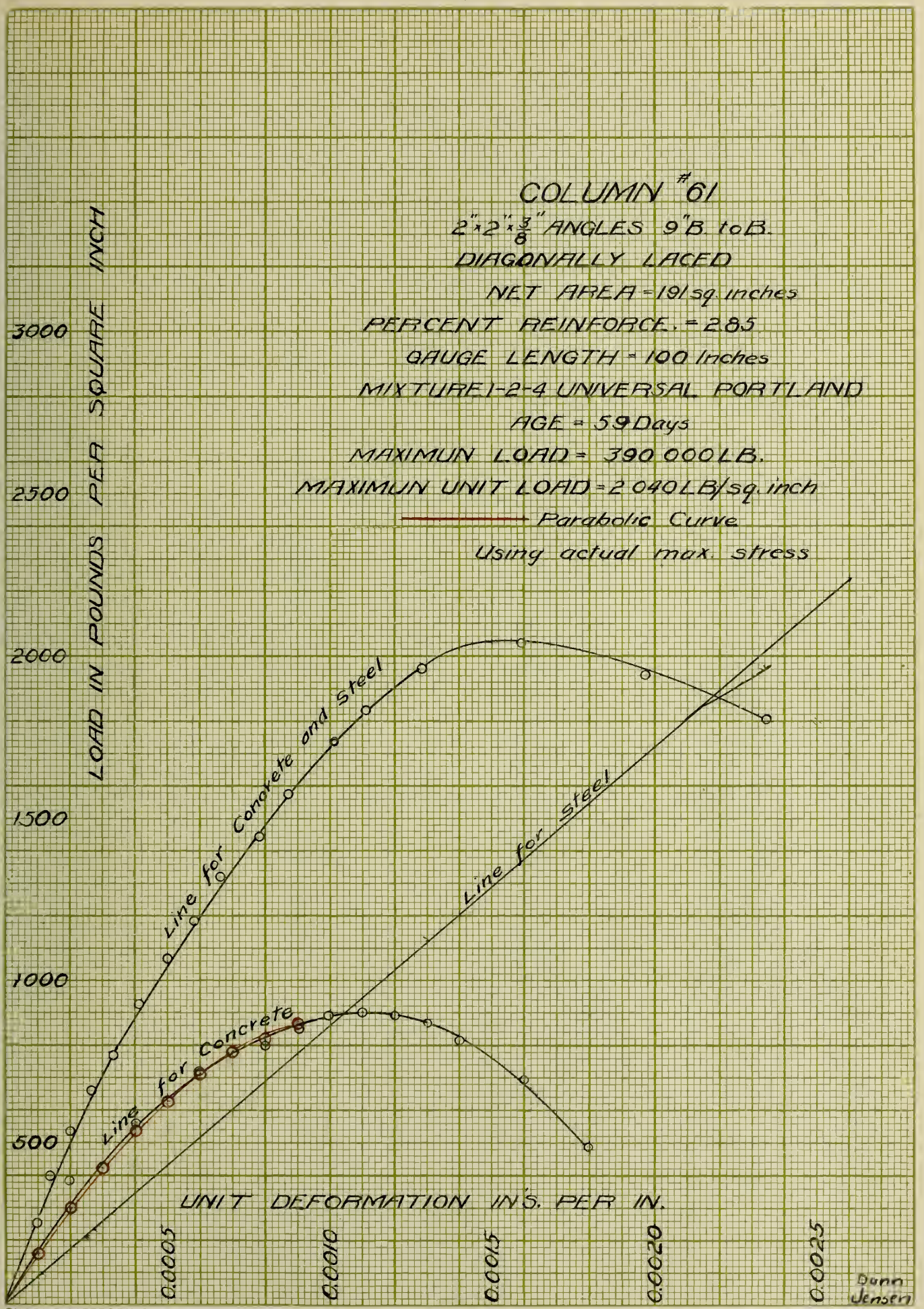
Fig. 13.

COLUMN 61.

Observed and Computed Data.

Length, 10 ft. 0 in.
Gauge Length, 100 in.
Cross-section, 14 1/2 x 14 in.
Size of Angles, 2 x 2 x 3/8 in.
Mixture, 1 - 2 - 4.
Age when tested, 59 days.
Cement, Universal Portland.
Distance, Back to Back 9 in.

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in	per 1 in.
6500	34	.0000	.00000
25000	131	.0035	.00004
52000	272	.0070	.00007
75000	393	.0136	.00014
100000	523	.0198	.00020
125000	655	.0259	.00026
150000	785	.0332	.00033
175000	917	.0407	.00041
201000	1055	.0493	.00049
225000	1178	.0584	.00058
250000	1310	.0663	.00066
275000	1440	.0779	.00078
300000	1570	.0871	.00087
331000	1730	.1012	.00101
350000	1830	.1112	.00111
375000	1960	.1284	.00128
390000	2040	.1589	.00159
370000	1940	.1970	.00197
345000	1800	.2340	.00234
320000	1670	.2904	.00290
305000	1595	.3282	.00328



EUGENE DIETZGEN CO., CHICAGO.

Fig. 14.

COLUMN 63.

Observed and Computed Data.

Length, 10 ft. 0 in.
Gauge Length, 100 in.
Cross-section, 14 x 14 in.
Size of angles, 3 x 3 x 1/2 in.
Mixture, 1 - 2 - 4.
Age when tested, 60 days.
Cement, Universal Portland.
Distance, Back to Back, 9 in.

Load Pounds.	Unit Load lbs. per sq. in.	Longitudinal Deformation. per 100 in.	per 1 in.
9000	50	.0000	.00000
25000	135	.0025	.00003
51000	276	.0067	.00007
75000	405	.0119	.00012
101000	550	.0174	.00017
126000	680	.0224	.00022
152000	820	.0279	.00028
176000	950	.0338	.00034
201000	1085	.0395	.00040
226000	1220	.0458	.00046
252000	1360	.0526	.00053
277000	1500	.0585	.00059
302000	1640	.0633	.00063
350000	1900	.0788	.00079
375000	2030	.0871	.00087
400000	2160	.0984	.00098
425000	2300	.1283	.00128
434000	2340	.1553	.00155
434000	2340	.1900	.00190
420000	2270	.2709	.00271
400000	2160	.3593	.00359
391000	2110	.4735	.00474

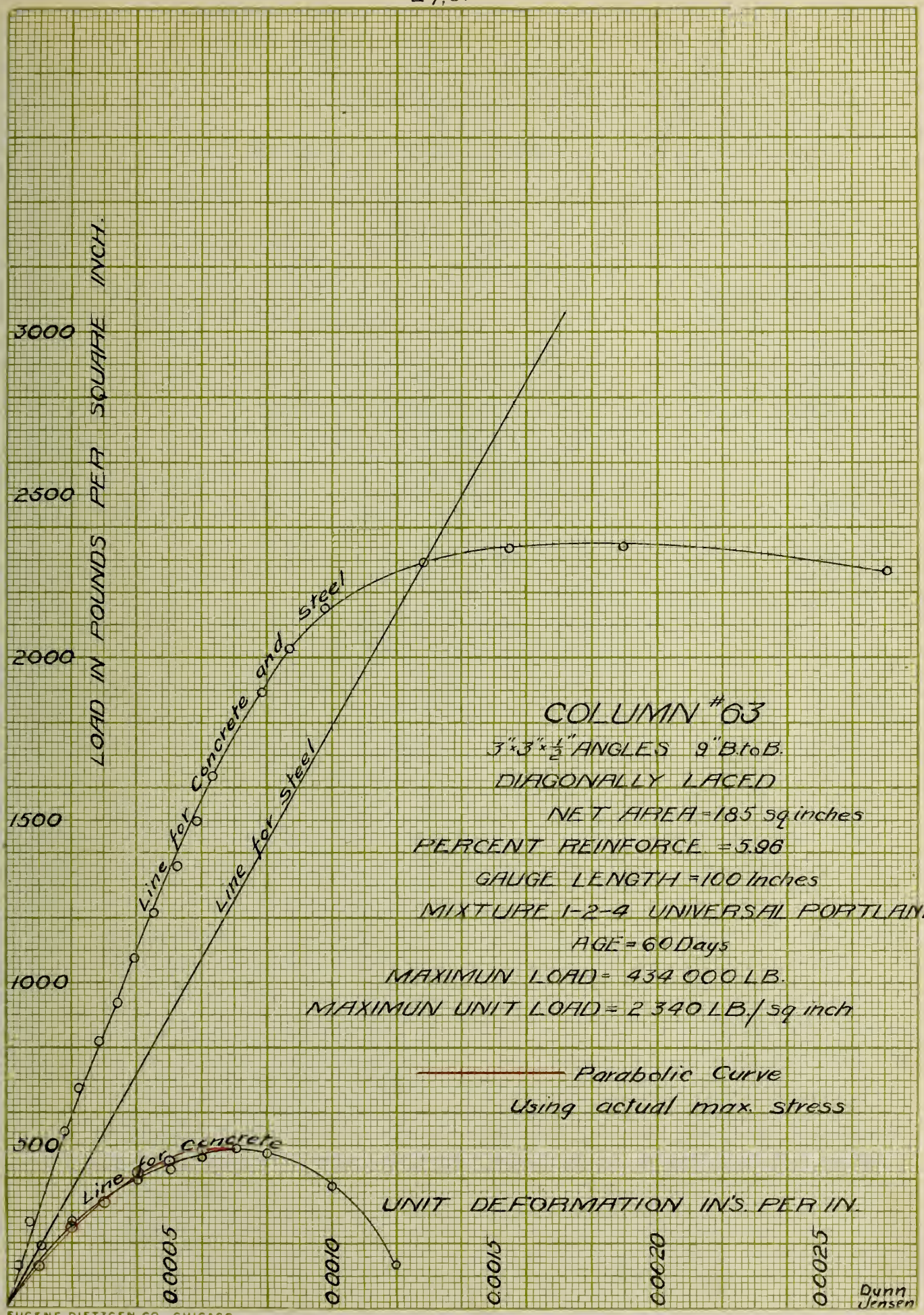

LOAD IN POUNDS PER SQUARE INCH.
3000
2500
2000
1500
1000
500
Line for concrete and steel
Line for steel
Line for concrete
COLUMN #63
3"×3"×½" ANGLES 9" B. to B.
DIAGONALLY LACED
NET AREA = 18.5 sq. inches
PERCENT REINFORCE = 5.96
GAUGE LENGTH = 100 inches
MIXTURE 1-2-4 UNIVERSAL PORTLAND
AGE = 60 Days
MAXIMUN LOAD = 434 000 LB.
MAXIMUN UNIT LOAD = 2 340 LB./sq inch
Parabolic Curve
Using actual max. stress
UNIT DEFORMATION IN'S. PER IN.
0.0005
0.0010
0.0015
0.0020
0.0025
Dunn
Jensen
EUGENE DIETZGEN CO., CHICAGO.

Fig. 15.

TABLE NO. 8.

2. Summary of Column Tests.

Col. No.	Reinforcement. Kind.	Reinforcement. Per-cent.	Maximum Load Total pounds.	Maximum Load lbs. per sq. in. of Net. Section.	Age Days.	Manner of Failure.
31.	4-3x3x1/4" Angles laced. 12" B to B	3.03	438000	2310	63.	Crushed in upper two ft. along angles. Angles buckled between lacing bar connection in upper two feet.
32.	4-3x3x1/2" Angles laced. 12"Bto B.	5.96	400000	2160	63.	Crushed in upper two ft. along line of angles. Angles buckled between lacing bar connections in upper two feet.
34.	4-3x3x1/2" Angles Hor. tied. 12"B to B.	5.96	570000	3080	53.	Crushed in upper two ft. along line of angles. Angles buckled between lacing bar connections in upper two feet. One rivet sheared off two ft below top.
40.	4-2x2x3/8" Angles laced. 9"B to B.	2.85	419000	2200	64.	Crushed in lower two ft. along line of angles. Angles buckled between lacing bar connections in lower two feet.
45.	4-3x3x1/2" Angles laced. 9"BtoB.	5.96	495000	2670	65.	Crushed in upper three ft. along line of angles. Angles buckled between lacing bar connections in upper three ft.

Table No.8 (con't.)

Col. No.	Reinforcement. Kind.	Per-cent.	Maximum Load. Total pounds.	lbs. per sq. in. of Net. Section.	Age days.	Manner of Failure.
48.	4-3x3x1/4" Angles laced. 12"BtoB.	3.03	440000	2310	65.	Crushed in lower half of angles. Angles buckled between lacing bar connections in lower three ft. Crushed slightly in upper foot.
52.	4-3x3x1/2" Angles laced. 12"BtoB	5.96	444000	2400	61.	Crushed in lower three ft. along line of angles. Angles buckled between lacing bar connections in lower three feet.
56.	4-3x3x1/2" Angles Hor. tied. 12"BtoB	5.96	480000	2600	59.	Crushed in lower three ft. and in middle along line of angles. Angles buckled between lacing bar connections at places of crushing.
61.	4-2x2x3/8" Angles laced. 9"BtoB,	2.85	390000	2040	59.	Crushed in lower third. Angles buckled between lacing bar connections in lower third.
63.	4-3x3x1/2 Angles laced. 9"BtoB.	5.96	434000	2340	60.	Crushed in middle third. Angles buckled between lacing bar connections in middle third.

TABLE NO. 9.

Summary.

3. Tests of 6 inch Cubes.

Cube No.	Age at Test. Days.	Maximum Load. Total lbs.	Lbs. per sq.in.	Mean lbs. per sq.in.	Maximum Stress in Concrete of Corresponding Column.
	70	94700	2630		
31	70	98100	2725	2572	1290
	70	84900	2360		
	69	77800	2160		
32	69	76200	2115	2125	970
	69	75600	2100		
	63	59700	1660		
34	63	76100	2110	1885	1010
	63	33600	Not set up well		
	84	93200	2590		
45	84	79700	2215	2460	1010
	84	92700	2570		
	74	62700	1740		
48	74	82000	2280	2027	1280
	74	74200	2060		
	64	69200	1923		
52	64	68600	1905	1844	680
	64	61000	1695		
	59	79400	2205		
56	59	94600	2630	2682	520
	69	115600	3210		
	61	61700	1715		
61	61	71300	1980	1818	900
	61	63300	1760		
	64	39200	1090		
63	64	39500	1100	1107	490
	64	40800	1130		

No Cubes tested for Column No. 40.

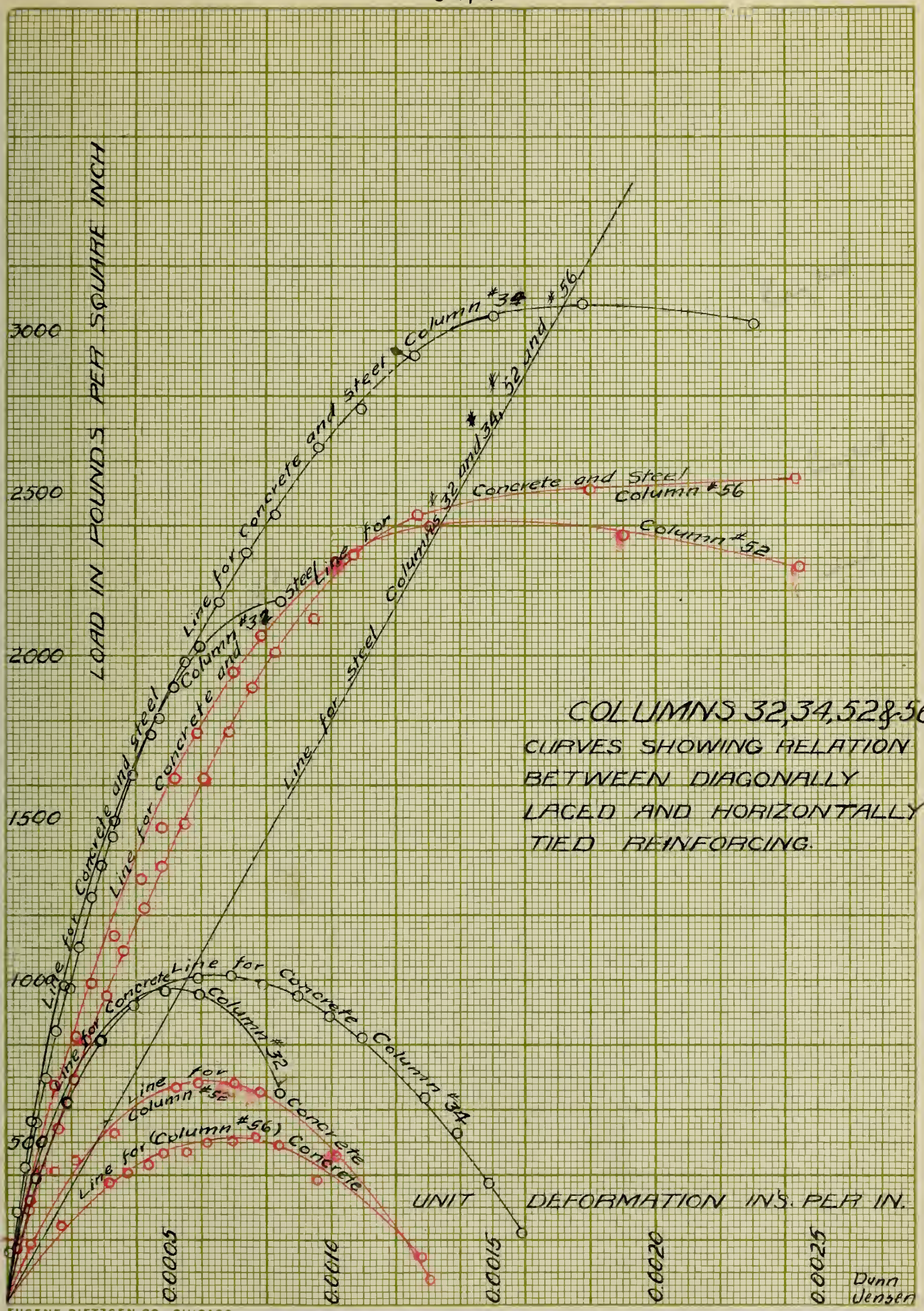
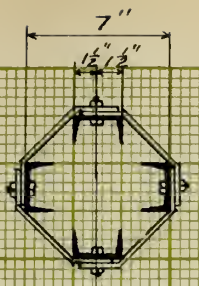
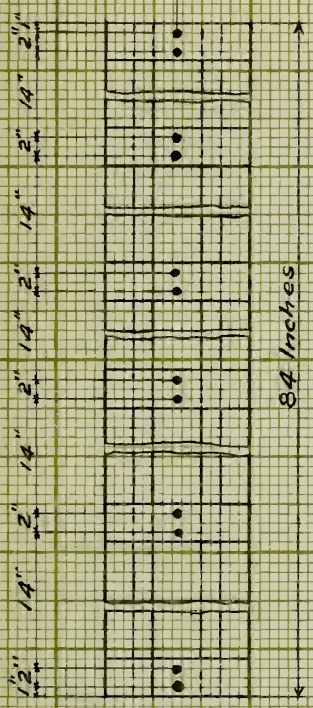

LOAD IN POUNDS PER SQUARE INCH
3000
2500
2000
1500
1000
500
Line for Concrete and steel Column #34
Line for Concrete and steel Column #56
#32 and #34
#32 and #34
Concrete and Steel Column #56
Column #52
Steel Line for Columns
Line for steel Columns
Line for Concrete and steel Column #32
Line for Concrete and steel Column #34
Line for Concrete Column #32
Line for Concrete Column #34
Line for Column #52 Concrete
Line for (Column #56) Concrete
COLUMNS 32,34,52&56
CURVES SHOWING RELATION BETWEEN DIAGONALLY LACED AND HORIZONTALLY TIED REINFORCING.
UNIT DEFORMATION IN'S. PER IN.
0.0005
0.0010
0.0015
0.0020
0.0025
Dunn
Jensen
EUGENE DIETZGEN CO., CHICAGO.

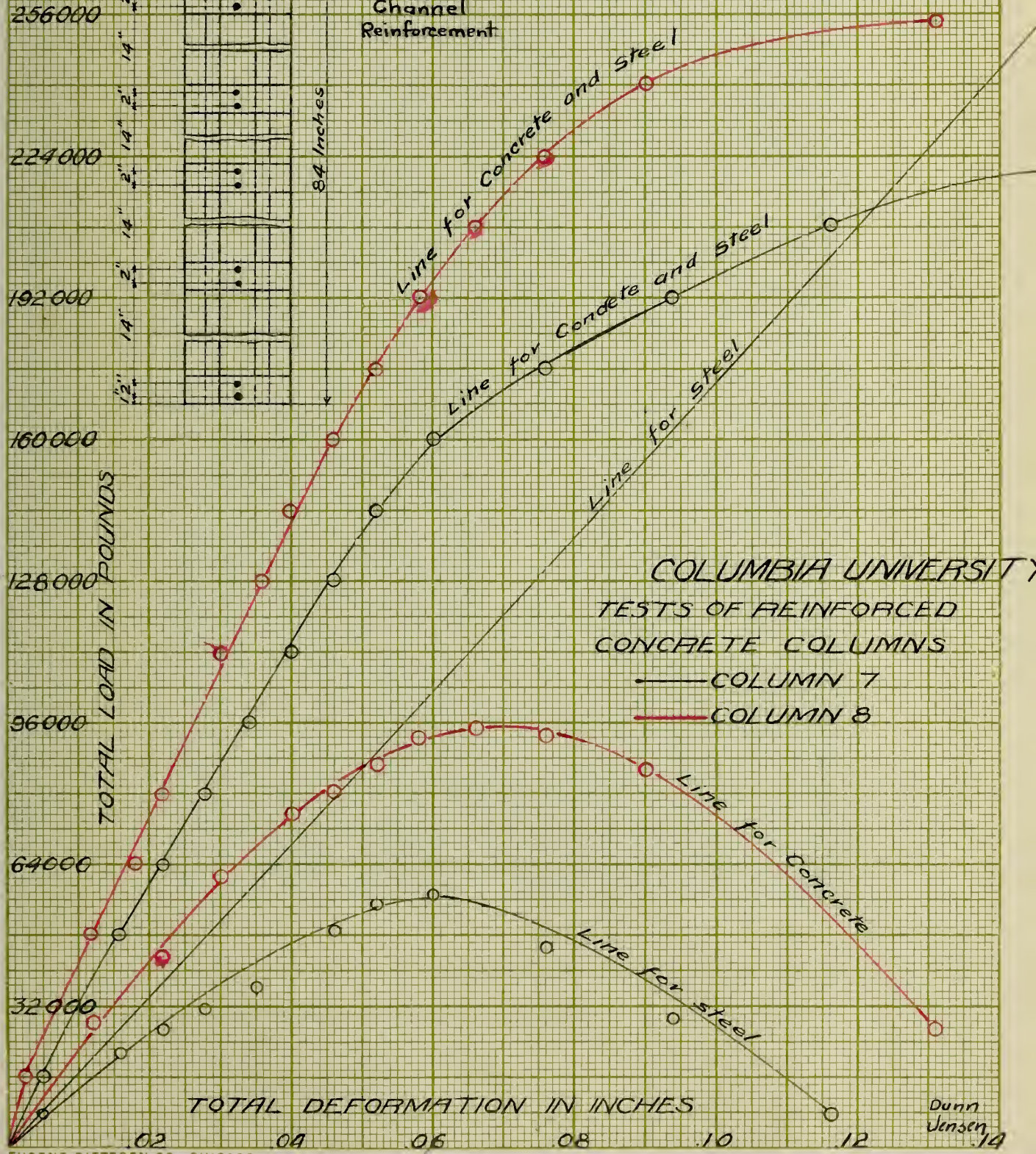
Fig. 16.



4 Channels 3"-4#
Battens 4" × ¼"
Bolts ½"

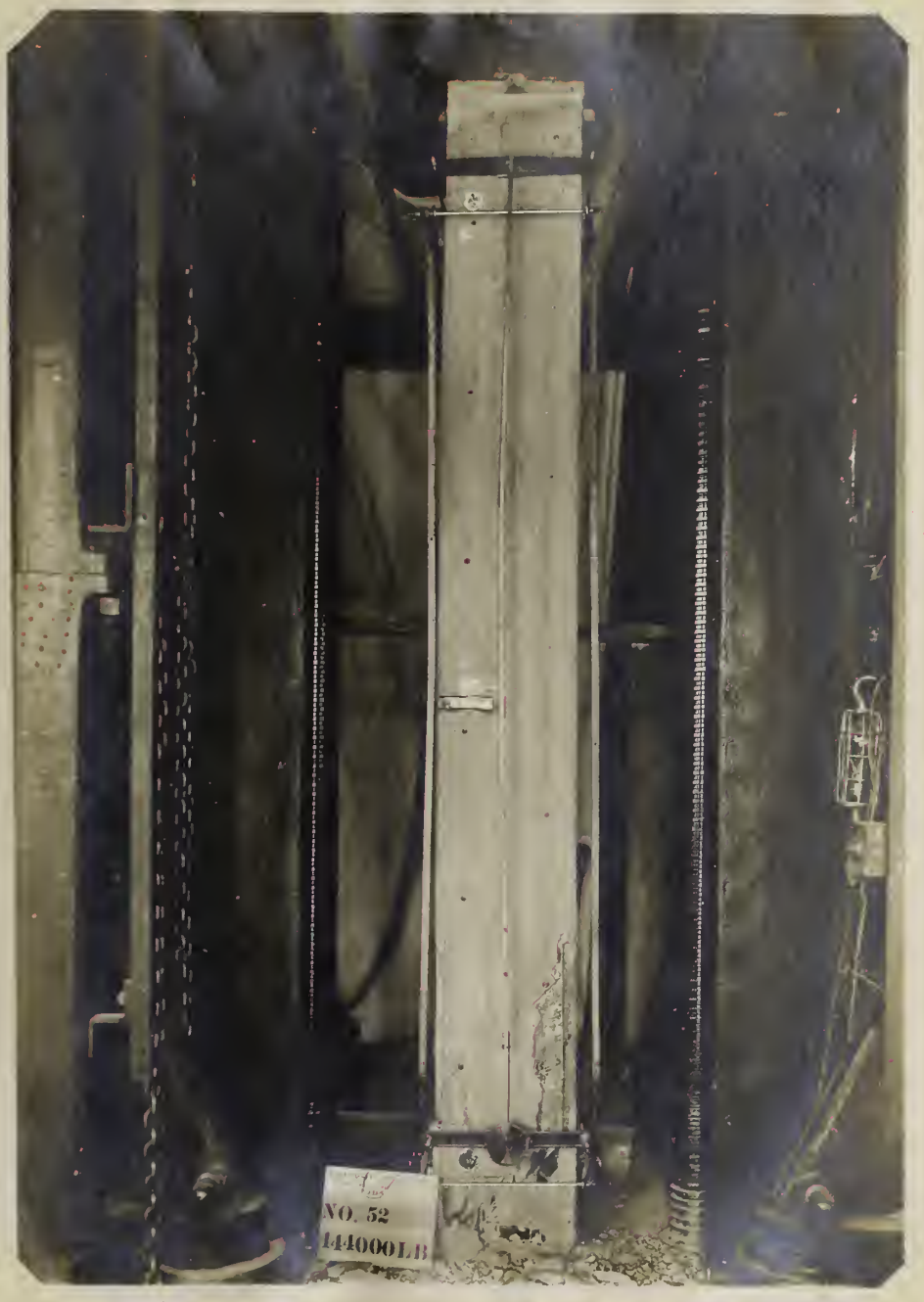


Sketch of Channel Reinforcement



EUGENE DIETZGEN CO., CHICAGO.

Fig 17



View showing instruments in place, Column 52 after initial failure.

Fig. 18.

View showing Column 40 after failure.

Fig. 19.

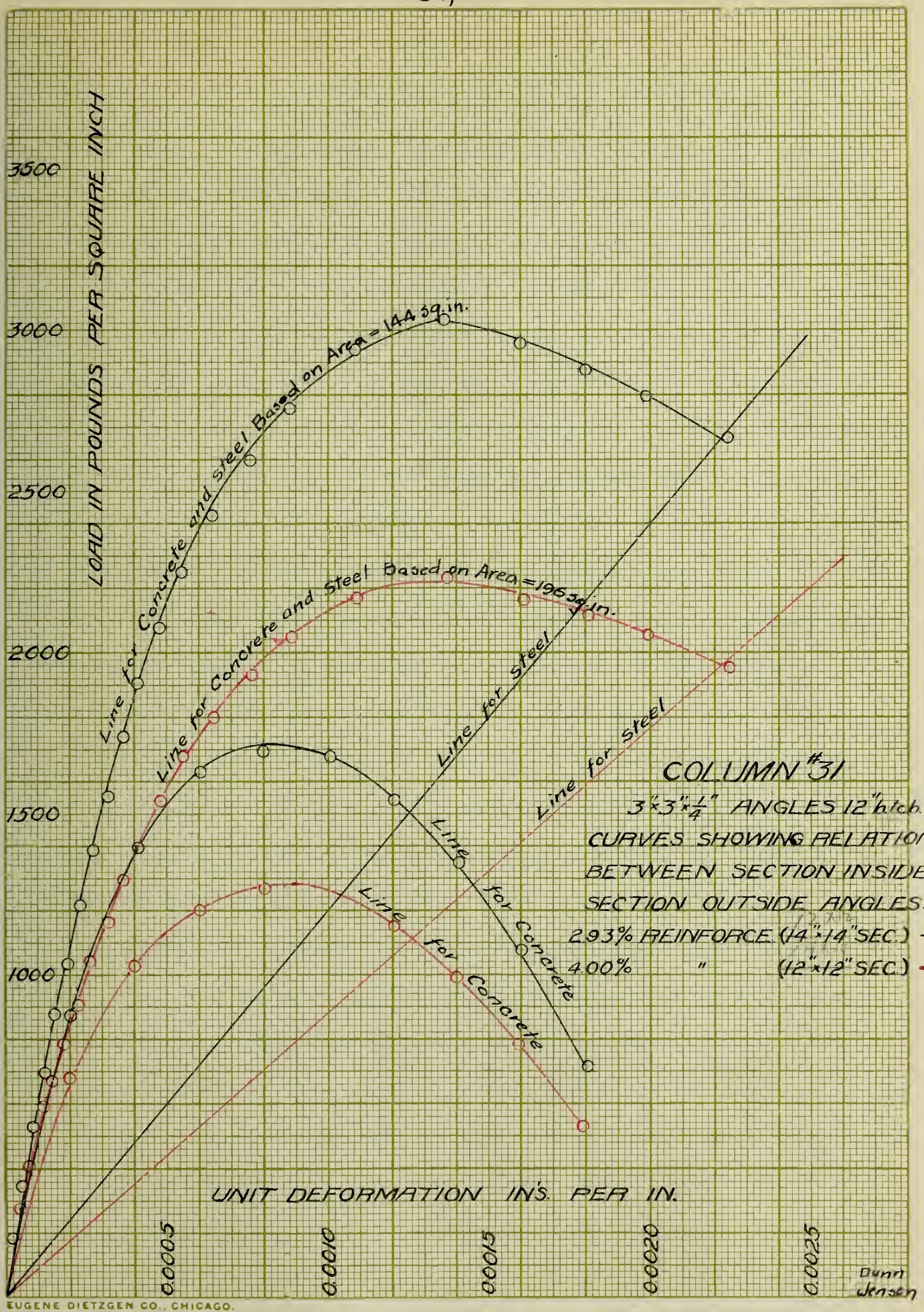

LOAD IN POUNDS PER SQUARE INCH
3500
3000
2500
2000
1500
1000
Line for Concrete and Steel Based on Area = 144 sq. in.
Line for Concrete and Steel Based on Area = 196.39 sq. in.
Line for Steel
Line for Steel
Line for Concrete
Line for Concrete
COLUMN #31
3"×3"×1/4" ANGLES 12" b/cb.
CURVES SHOWING RELATION
BETWEEN SECTION INSIDE,
SECTION OUTSIDE ANGLES.
2.93% REINFORCE (14"×14" SEC.)
4.00% " (12"×12" SEC.)
UNIT DEFORMATION IN'S. PER IN.
0.0005
0.0010
0.0015
0.0020
0.0025
Dunn
Jensen
EUGENE DIETZGEN CO., CHICAGO.

Fig. 30

PART VI.

Discussion of Column Tests.

Table No.(8) gives the age of columns when tested, maximum load, kind of reinforcement, and manner of failure.

1. Phenomena of Column Tests.

Most of the columns failed in either the top or bottom third of their length, only one failing near the center. With the first three columns the bearing of the upper plate was directly upon the one inch thickness of concrete that had been placed on top of the angles. The failure of these columns occurred in the upper third portion and , in fact, quite early in the test there occurred some slight cracking in the column immediately under the plate, indicating failure of the concrete. With the seven succeeding tests, however, this upper inch or two of concrete was removed and the bearing plate was placed directly upon the top of the angles, as was described in a previous chapter. With these columns the failure occurred, in every case but one, in the lower one third portion, and in no case was there any indication whatever of failure in the upper one third.

In most cases, little warning in the way of cracks or sound was given before the maximum load was reached. The failure in each case, however, when once begun was rapid and complete. In some cases, as will be noted, the load after reaching a maximum dropped off rather slowly indicating that the column was acting somewhat as a banded column.

The following notes show the principal features of the tests of individual columns. These columns are given in the order in which they were tested.

Column 31 took a maximum load of 438000 pounds with a total deformation at that load of .130 inches. At a load of 375000 pounds the concrete underneath the upper bearing plate began breaking off. At 425000 pounds vertical cracks were observed extending along the rivet lines of the angles in the upper one-third portion of their length. Immediately after this, at a load of 438000pounds the column showed marked signs of ultimate failure and the load began to gradually drop off. After about 5 minutes running the load had only dropped to 255000 pounds. At this stage nearly all of the concrete outside of the angles had peeled off in the upper three feet of the column. The angles also began to buckle badly. In every case the buckling occurred between lacing bar connections.

Column 32, though made up of heavier angle reinforcement, behaved very much the same as did Column 31. The maximum load taken was 400000 pounds and the total deformation at this load was .0854 inches. The deformation increments seemed to be smaller for corresponding loads than they were in Column 31. This column also stood up well and no apparent signs of failure occurred until a load of 375000 pounds was reached. At this load cracking along the edges of the angles in the top portion was observed . At first only the very top

few inches were cracked but as the load was applied the cracking rapidly progressed down the column until at a load of 400000 pounds the cracks extended a distance of two and one-half feet. On further application of this load these cracks continued to advance down the column until they extended two and one-half feet more, or a total of 5 feet. Further application of the load resulted in shearing off one of the rivets of the angles and the lacing and angles were buckled badly. The appearance of this column when the load was finally taken off was almost identical with the appearance of Column 31 when it was taken from the machine.

Column 34, also made up of 3 x 3 x 1/2 inch angles differed from Column 32 in that the angles were horizontally tied at intervals of 12 inches instead of being laced. This column acted very much the same as Column 32 up to a load of 400000 pounds, the maximum load for Column 32. At this load Column 34, though showing cracks along the inner edge of the angles as did Column 32, still seemed sound and strong. On the application of more load the cracks mentioned above became more prominent and gradually worked their way down the column until at a load of 570000 pounds, the maximum load, they extended about four feet. At this load the outer concrete began to peel off and the continued application of the load resulted in more peeling off of this concrete. With this column, however, the peeling was not confined to the narrow limit of three feet as

was the case with No. 32, but it was more or less general throughout the length of the column. A particularly bad section of this peeling occurred at the upper extremity and a similar section at the lower extremity.

Column 40 varied considerably from the three previous columns in the size of the angles and their distance back to back. The angles used were 2 x 2 x 3/8 inches placed 9 inches back to back. This small distance back to back resulted in a very thick shell of concrete outside the angles of about four inches. This thick shell of concrete, when combined with the interior concrete it was thought might cause the column as a whole to act as a plain concrete column. The experiments to a great extent bore out this idea. The maximum load was 419000 pounds with a total deformation at this load of .145 inches. This column acted similiar to the previous columns except that it took its maximum load before any sign whatever of failure occurred. At this point the load fell off suddenly and without warning to 377000 pounds and cracks appeared in the lower portion of the column. This was the first column that failed in the lower portion and it was also the first column in which the bearing of the upper plate was directly upon the angles. This was a coincidence that was worth some attention. It seemed possible that the difference in the bearing of this column and the previous columns might account for this difference in the manner of failure. Further tests of columns with plates bearing simil-

arly seemed to substantiate this idea.

Column 45 acted similar to Column 40 except for a slight difference in its manner of failing. The maximum load was 495000 pounds with a total deformation of .130 inches at this load. A deflectometer reading to .01 of an inch was put on this column (shown in Fig. 18) No deflection whatever was observable until the maximum load was reached and even then the deflection was slight. No cracking occurred until the maximum load was reached. At this point the load suddenly dropped to 450000 pounds and cracks through the middle portion of the column were observable. This column also remained sound in the upper portion of its length.

The behavior of Columns 48, 52, 56, 61, and 63 under stress ~~were~~ was similar to those of their duplicates Nos. 31, 32, 34, 40, and 45 respectively except that where the columns tested earlier in the year failed in the upper portion these had their failure in the lower portion. There were some slight discrepancies in loads and deformations which are well shown in the accompanying stress- deformation diagrams.

2. General Discussion of Tests.

A few remarks at this point as to the reason for the rather peculiar way in which the first three columns failed may not be out of place. These three columns had the bearing of the load upon the top 2 inches of concrete, this concrete, as previously explained, being placed upon the top of the angles.

The entire load was thus taken at first by this 2 inches of concrete. The stress from this concrete after reaching the steel was transmitted to it by the bond which existed between the concrete and the steel. The depth to which the stress would have to go before it would transmit to the steel its proportionate stress would be calculated in the following way. Take Column 32 for example. The angles 3 x 3 x 1/2 inch which have an exterior perimeter for each angle of 11 inches. Total exterior perimeter of angles = 4 x 11 = 44 inches. Assume the adhesive strength of the concrete equal to 300 pounds per square inch. Then for every inch in depth of the steel angles a stress of 44 x 1 x 300 pounds = 13,200 pounds would be transmitted to the steel. For this column the maximum stress in column was 400000 pounds. But the unit stress for the concrete as shown by the concrete curves (Fig. 7) is only 1000 pounds per square inch. This multiplied by the area of the concrete (185 sq. in.) gives only 185000 lbs. 400000 pounds - 185000 pounds = 215000 pounds = load carried by adhesive bond of concrete. Column carries 13200 pounds bond per linear inch in length of column.

$$\frac{215000}{13200} = 16 + \text{inches.}$$

This would indicate that a stress large enough to cause failure in the concrete existed in top 16 inches provided no load was transmitted to the steel by direct compression. The manner of failure showed this to be the case, for in every instance

when the column was loaded as described above the failure was first observed in the first few inches. When the bearing was directly upon the angles, however, a unifrom distribution of stress for increment deformations was possible.

3. Cube Tests.-

Table No. (9) gives data of the age at test and of the compressive strength of the 6 inch cubes used in conjunction with the column tests. These cubes were made from the same batch, at the same time, and under the same conditions as were the correspondingly numbered columns. The tests were made merely as a check upon the concrete that was used. The strength of the cubes in compression is no index of what the strength of the concrete in the correspondingly numbered columns should be except in a relative sense. Where a cube takes a low maximum stress compared with other cubes similarly made it is to be expected that the concrete in the corresponding column will take a low maximum stress.

A comparison of the ultimate strength of the cubes and of the concrete in the reinforced columns shows a great discrepancy. This discrepancy is undoubtedly due to two causes. 1st the restraining influence that the bearing block of the machine head exerts upon cubes must enter materially into the stresses produced in them, and 2nd the concrete in the cubes had been placed under more favorable conditions than that in the reinforced concrete columns. The concrete in the columns

had to be poured into a form about the steel angles and could not be tamped as thoroughly as was necessary to insure uniformity of mixture. The column being no stronger than the weakest aggregation in its make-up would fail at this particular point and the failure as noted would not necessarily be the maximum strength of the column, but merely the maximum strength of the weakest spot in the column.

4. Stress-deformation Diagram.

In Fig. 6 to 15 , are given the stress deformation diagrams. These diagrams represent the observed unit loads and the corresponding deformations or shortening for the column tested. The ordinates represent the loads per square inch of net cross section area of the column. An explanation of what constitutes net cross section area will be given later on in this article. The abscissas represent the unit deformation, or shortening per unit of length, determined from the extensometer readings for a gauged length of 100 inches. These values are the averages of the readings that were taken on the four faces of the columns. In general, there was some variation in the four readings due to unequal deformation in the column but these variations were slight and usually consistent in that where one extensometer read high the opposite extensometer read low, the average giving the mean or true deformation.

In these diagrams the amount of deformation is

calculated by using as the zero reading the extensometer reading at the initial load, usually 6000 or 7000 pounds. The deformations are thus independent of any change or set which this load may have produced in the concrete.

The net cross section area used in determining the unit stress is the total cross section area of the column minus the cross section area of the four angles. The total cross section area of the column was arbitrarily taken as 14 x 14 = 196 square inches. This value was used because the column, though varying somewhat in its cross section through out its length, was designed for, and averaged up, a section of these dimensions. The cross section area of the angles only was taken, no attempt being made to deduct the area of the lacing bars which might enter into the cross section under consideration. For the first two columns several diagrams were made one of which is shown in Fig. 20 to determine what should be considered as the cross section area of the column. First the interior area of the steel was considered as being the only concrete area that took any stress, the outer 2 inch being considered a mere shell. This however gave such large stresses for the concrete alone that it was deemed to be in error. Then curves were drawn considering the entire 14 x 14 cross section as concrete and basing the percentage of reinforcement on this cross section. This led to erroneous results for proper allowance had not been made for the stress taken by the steel alone. The results

were then calculated assuming the area of the cross section as the total cross section minus the area of the four angles in this cross section. These results were deemed correct and the final and accepted curves were drawn from them.

First the stress deformation curve was drawn for the concrete and steel combined. Then the steel line was drawn, after which the stress in the concrete proper was determined by deducting from the combined curves the stress in the steel for corresponding deformations. The following line of reasoning was used in arriving at these results.

Let A = Area of concrete. P_c = Unit stress in concrete

A' = Area of steel. P_s = Unit stress in steel.

$A + A'$ = Total area. e_s = Unit deformation in steel.

e_c = Unit deformation in concrete.

P = Total stress = $A P_c + A' P_s$, but according to the theory of proportionality of stress we know that for equal deformations the concrete and steel will take stress in proportion to their moduli of elasticity, i.e.

$$\frac{E_s}{E_c} = \frac{\frac{P_s}{e_s}}{\frac{P_c}{e_c}}, \text{ since } e_s = e_c, \quad \frac{E_s}{E_c} = \frac{P_s}{P_c}.$$

Let p = Net percentage of reinforcement. $= \frac{A'}{A}$

$\therefore A' = p A$

Since $E_s = \frac{P_s}{e_s}$, $\therefore P_s = E_s e_s$

$$\therefore A' P_s = p A e_s E_s$$

$$\therefore P = A P_c + A p E_s e_s$$

$$\frac{P}{A} = P_c + p E_s e_s = \text{Combined unit stress.}$$

For determining the "line of steel" a modulus of elasticity for steel of 30,000,000 pounds per square inch was used. This is the commonly accepted value for structural steel. A unit deformation was then decided upon and the unit stress for this deformation was calculated from the formula

$$P_s = 30{,}000{,}000 \times e_s$$

1. $P = A P_c + A p E_s e_s = A P_c + A' P_s$

2. $A p E_s e_s = A' P_s$

3. $\frac{A' P_s}{A} = p E_s e_s$

4. But $\frac{A' P_s}{A}$ = total stress in steel ÷ area of concrete = stress per square inch taken by the steel if it were considered as distributed over the entire cross section of the concrete.

5. Equation 4. is the equation of the line for steel.

To illustrate the method of drawing the line for steel

$$\text{Let } e_s = 0; \text{ then } \frac{A' P_s}{A} = 0$$

$$\text{Let } e_s = .001 \text{ ; then } \frac{A' P_s}{A} = p \times 30\ 000\ 000 \times .001 = 30\ 000\ p$$

A straight line through the two points as thus determined will be the line of steel. The stress in the combined concrete and steel being known, the stress in the steel having been calculated, the

stress in the concrete was then determined by subtracting the latter from the former.

The first thing that attracts our attention when looking at the stress-deformation diagrams is their symetrical appearance and the consistent manner with which they follow the points plotted. The curves are regular up to the point of maximum stress when their behavior varies with the various kinds of reinforcement; some gradually droping off, some running nearly horizontal, and some dropping abruptly from this point. The symetrical appearance of the curves might be explained in the following way. If the columns were steel alone the stress-deformation curve would have been a straight line up to the elastic limit of the steel. If the column were concrete alone the curve would have bee a parabola up to the point of maximum stress, about 1750 pounds per square inch. Now, if the concrete were homogeneous and well bonded to the steel it would be expected that the resulting curve would be a modification of the parabola due to the straight line curve of the steel. The symetry of the curve would be an index of the unity of action of the concrete and steel. The curves unquestionably do show this unity of action.

One thing that caused the writers some concern when they plotted up the curves was the fact that the maximum load shown by the deduced concrete curve was exceedingly low and occurred at a deformation below what would have been the

case, had the column been plain concrete. It seemed strange that the columns should act this way and some fear was entertained as to the accuracy of the observation that had been taken. A careful comparison was then made between the deduced concrete curve as plotted and the values for the stress ~~s~~ es and deformations as noted. The discovery was then made that where the concrete curve showed a maximum value for a certain deformation a reference to that deformation in the note books showed that very near this point the appearance of cracks in the column ~~were~~ was first noted. The values for the maximum stress in the deduced concrete curve ~~were~~ are very likely correct. Though these results are exceptionally low it seems that they must be accepted as the true values for the concrete when used in conjunction with the reinforcement under investigation in this thesis. The concrete is not apparently in a position to be most efficient.

To determine whether the concrete curve was probably correct or not another check on the curves was made. A curve was drawn from an assumed point to the origin using the parabolic relation for the stress and deformation. The assumed point taken was a stress of 16000 pounds per square inch and a deformation of .00125 inches. This is a mean maximum value for 1 - 2 - 4 concrete, 60 days old. After this concrete curve was drawn a steel line was constructed by deducting from the combined steel-concrete curve the concrete curve just drawn. This steel line instead of being a straight line, as it would necessarily have to be with the low stresses and deformations used, developed

into a curve instead of a straight line. This showed beyond question that the value assumed for concrete was in error.

Finally, to go a little farther toward proving the assumption that the deduced concrete curves shown in the stress-deformation diagrams (Fig. 6 toFig. 15) are correct a parabola through the origin was drawn with its vertex at the point of maximum stress and deformation as shown in the concrete curve. This parabola in most cases followed the concrete curve quite closely and, in several cases, almost exactly. The usual assumption that is made regarding the action of plain concrete under stress is that the stress- deformation curve is a parabola. The fact that the concrete curve deduced from the combined curves of the stress-deformation diagrams closely approximates a parabola tends to prove quite conclusively that these curves are correct.

In Fig. 16 a comparison is made of the stress- deformation curves of four columns all of which were of the same cross section area, of about the same age, and had the same size and kind of angles for reinforcement. They were only different in the method with which these angles were tied together. Columns 34 and 52 were tied by means of diagonal lacing while Columns 32 and 56 were tied by means of horizontal ties. These two were the only columns in which horizontal ties were used, diagonal lacing being used on all the others. When the first tests of columns with these two types of reinfor-

cement were made the horizontally tied column showed up as the stronger and better column, but when the next two tests were made this marked difference in the two types of columns was not so apparent. For both types the concrete alone took about the same stress but the horizontally tied column differed from the laced column in the manner of its failure. The curve for the laced column after reaching its maximum dropped off quite suddenly, much the same as the curve for a plain column or column with vertical rods for reinforcement would act. With the horizontally tied column this sudden drop in the curve was absent. The curve after reaching a maximum bent sharply into a more or less true horizontal line. With one of these two columns the line continued at a slight angle above the horizontal, with the other the angle was below. In this respect the curves bore some, though not a very striking, resemblance to the curves of the hooped type of column. It seemed evident that there was some banding effect in the reinforcement.

5, Comparison with Columbia University Tests.-

A comparison with two tests made by Proffessor W. H. Burr of Columbia University will now be made. The tests were made on two concrete columns reinforced with channels. The reinforcement was of the type and of the dimension shown in the sketch (Fig. 17). The columns were only 7 feet long and 7 inches back to back of channels but as these were the only tests available they will have to be used in this comparison. The section as shown in Fig.17 was of octagonal shape.

No data other than that afforded by the accompanying sketch##, Fig. 17 ~~and Fig.~~ was furnished so that certain assumptions had to be made. First it was assumed that the concrete was placed only inside of the channels, no outside shell of concrete being used. The concrete was assumed to be of 1-2-4 mixture, 60 days old. With these assumptions the following calculations were made.

Area of 3" - 4 # channels = 1.19 sq. in.

4 channels = 4 x 1.19 = 4.76 sq. in. (cross section of steel).

Area of cross section = Area of octagon with short diameter of 7" = .828 x 7^2 = 40.60 sq. in.

Area of net section = 40.60 - 4.76 = 35.84 sq. in.

$$\text{steel} = \frac{4.76 \times 100}{35.84} = 13.3\ \%$$

From formulae derived earlier in this article the steel line and concrete curve were drawn.

For a deformation of .001 inches the stress for the steel equals 30 000 x .133 = 4000 pounds per square inch of concrete section.

Total stress in steel = 4000 x 35.84 = 143 400 lbs. Deformation of .001 inches per inch gives .084 inches for 84 in. From this point the steel line was drawn to the origin.

The concrete curve for column No. 7 shows a total maximum stress of 57500 pounds (unit stress = $\frac{57500}{35.84}$ = 1600 pounds

per square inch), and a total maximum deformation of .0560 inches (unit deformation = $\frac{.0560}{84}$ =.0007 inches per inch). The concrete curve for Column No. 8 shows a total maximum stress of 97100 pounds(unit stress = $\frac{97100}{35.84}$ = 2700 pounds per sq. in.), and a total maximum deformation of .07 in. (unit deformation = $\frac{.0700}{84}$ = .0008 inches per inch).

The values for the unit stress of both Columns 7 and 8 are considerably higher than those for the columns tested under this thesis. The unit deformation however, corresponds quite closely with the unit deformation of these columns. It is in this respect that these tests are valuable as a comparison. It is interesting to note that these columns, which are also reinforced with structural shapes, should take such low values for maximum deformation in the concrete. If the assumptions as to area and mixture of these columns are true there is additional reason for believing that this low maximum deformation for columns reinforced with structural shapes, either tied or laced, is a characteristic of such columns.

PART VII.
SUMMARY.

The number of tests specimens for each kind of reinforcement is not large enough to permit any absolutely definite conclusions to be formed as to the relative efficiency of the different types or percentages of reinforcement. From the tests at hand however, the following conclusions might be drawn.

1. - The average ultimate strength of these columns made of 1-2-4 concrete, 60 days old, reinforced with steel angles is as follows : 3" x 3" x 1/2" laced angles, 12" back to back, 3x3x1/2" laced angles,back to back,2550 lbs.per sq.in. 2390 pounds per square inch; 3" x 3" x 1/2" horizontally tied angles 12" back to back, 2850 pounds per square inch ; 3" x 3" x 1/4" laced angles 12" back to back, 2300 pounds per square inch; 2" x 2" x 3/8" laced angles 9" back to back, 2130 pounds per square inch. Based on the abovevalues the reinforcement is at its best when it is horizontally tied . The reinforcement also shows up to better advantage when the angles are laced 9" back to back than when they are laced 12" back to back. As was to be expected the columns were stronger the larger the angles used.

2.- The average ultimate strength of the concrete alone when used with this type of reinforcement is as follows: for columns with 3 x 3 x 1/2 inch laced angles 12" back to back, 840 pounds per sq. in.; 3 x 3 x 1/2 inch laced angles , 9" back to back, 750 pounds per square inch; 3 x 3 x 1/2 inch horizontally tied angles 12" back to back, 760 pounds per square inch; 3 x 3 x 1/2 inch laced angles, 12" back to back, 1280 pounds

per square inch; 2 x 2 x 3/8 inch laced angles 9" back to back, 1050 pounds per square inch. The above values placed the concrete in a rather bad light. The average value for 1-2-4 60 day, concrete as determined from an extensive series of tests made last year is 1730 pounds per square inch. None of the values given above show any where near this strength but are from 50% to 75% lower. The values are quite consistent, however, and though low, are probably correct. The concrete when used with this type of reinforcement is not in a position to be most efficient.

3.- The angles when laced showed none of the characteristics of band reinforcement. When horizontally tied however the banding effect of the reinforcement became quite apparent, but there was little or no added strength by reason of the banding.

4.- The failure in the concrete occurred at as much as 50 percent less deformation, than experiments have shown plain concrete columns would take.

5.- The failure of the column as a whole occurred at a deformation(except in the case of the horizontally tied columns) of .0013 inches per inch or there abouts. This is the average ultimate deformation for 1-2-4 concrete 60 days old and would tend to show that the entire column failed when the concrete itself failed. A curve drawn on this assumption developed a curved steel line which could not be and showed the assumption in error.

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